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Santa Clara Valley
Water District

GEOTECHNICAL INVESTIGATION REPORT
PERMANENTE CREEK FLOOD PROTECTION PROJECT
MOUNTAIN VIEW AND LOS ALTOS, CALIFORNIA
PROJECT NO. 26244001

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1 copy:	Mr. Saeid Hosseini Senior Project Manager Santa Clara Valley Water District

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation conducted by the Santa Clara Valley Water District (District) geotechnical engineering staff. This report will be utilized by the project team in preparation for the design plans and specifications and construction of the Permanente Creek Flood Protection Project (PCFPP). As shown in the Site Location and Vicinity Map, Plate 1, the project site is located in the cities of Mountain View and Los Altos, California. According to the 60 % Design Plans, project improvements will extend from Miramonte Avenue in Los Altos to downstream of Amphitheatre Parkway in Mountain View conveying 100-year design flows to the bay.

1.1 Project Description

The PCFPP is part of the 2000 voter approved Clean, Safe Creeks, and Natural Flood Protection Program. In accordance with this program, the proposed improvements to Permanente Creek shall protect approximately 1,664 parcels downstream of El Camino Real from a 100-year flood and prevent flooding of Middlefield Road and Central Expressway.

Permanente Creek flows south to north draining portions of Mountain View, Los Altos, Los Altos Hills, Cupertino, and unincorporated Santa Clara County. Water flows from the Santa Cruz Mountains in the south to the floor of the Santa Clara Valley and the San Francisco Bay in the north.

Permanente Creek forms in the upper watershed area at the confluence of the North Branch (Ohlone Creek) and South Branch Permanente Creeks. The major tributary to Permanente Creek is Hale Creek, which connects to Permanente Creek just south of El Camino Real. The majority of the flows from the upper Permanente watershed area are diverted to Stevens Creek through the Permanente Diversion Channel. Water flows in Permanente Creek, Hale Creek and the Permanente Diversion Channel are conveyed through box culverts and open channels.

The purpose of the project is to provide flood protection for homes, businesses, and other facilities in the cities of Mountain View and Los Altos by accomplishing the following objectives:

- Provide one-percent (100-year) flood protection to approximately 470 parcels in the Permanente Creek Watershed by December 2011.

A preliminary geotechnical investigation was previously performed by the District geotechnical engineering staff for the Permanente Creek Planning Study, and the results were presented in a draft report dated February 2, 2005. During the project design development process, we also provided preliminary recommendations in the earlier phases of design. This geotechnical investigation will supplement the results of our preliminary 2005 investigation and earlier recommendations by providing specific conclusions and recommendations for design of the various project elements.

The project elements include:

- Floodwalls north of Highway 101 on levee channels
- New levee north of Amphitheatre Parkway over the existing west bank levee
- Channel widening along reaches of Permanente Creek and Hale Creek (including new U-Frame channels and one box culvert replacement)
- A new diversion structure at the upstream end of Permanente Diversion

Floodwalls and Levee

Concrete floodwalls will be built on top of the existing levees north of Highway 101, raising the top of bank to the required freeboard elevation. The downstream end of the floodwall protection would be at the junction with Amphitheatre Parkway. Short sections of floodwall will be built at Mountain View Avenue along Permanente Creek and at Arroyo Road along Hale Creek. Downstream of Amphitheatre Parkway, the only flood control element will be a new levee built over the existing west levee bank.

Previous geotechnical investigations have indicated that the existing levees are generally in good condition. The majority of floodwall work would begin immediately upstream of Amphitheatre Parkway and continue to Highway 101. The floodwall heights, measured from existing ground, would range from one foot to a maximum of approximately 4 feet, with an average elevation of 2 feet. Typical floodwall construction procedures would include trench excavation, form work installation, concrete pouring, and backfill.

The new levee fill along the west bank will span approximately 415 feet downstream of Amphitheatre Parkway and ranges in height from zero to a maximum of approximately 2½ feet. Additional fill will be placed along the landside slope to build the new levee and likely will require the project team to acquire easements from the City of Mountain View and Google to cover the lateral extents of the constructed levee. Typical construction procedures would include keyway excavation, benching into the existing levee, and compacting borrow or import fill materials.

Additional floodwalls will also be needed along the existing maintenance ramps. Approximately 40 linear feet of floodwalls at the downstream end of Mountain View Avenue at Permanente Creek (between approximate C-Line Stations 249+90 and 250+30) and approximately 50 linear feet at the downstream end of Arroyo Road (between Hale Creek Stations 12+10 to 12+60). The floodwalls are needed to prevent eddies at the existing ramps.

Channel Widening – Permanente Creek

This work involves deepening and enlarging the current concrete channel in Permanente Creek from just upstream of Mountain View Avenue to the confluence with Hale Creek. The planned design would replace the most badly degraded part of the concrete channels built in the 1960s, thereby addressing the most urgently needed part of the concrete channel restoration. It would also increase channel capacity upstream of McKelvey Park so that the full one-percent flows can reach the McKelvey Park flood detention site.

Due to the very limited right-of-way available and the deteriorated condition of the existing channel, underpinning will be needed prior to removal of the existing channel invert slab. The new channel construction would lower the invert elevation upstream of Mountain View Avenue. The upstream grade would be steepened so that it would conform to existing invert at the Hale Creek confluence.

Channel Widening – Hale Creek

This work involves increasing the capacity of some downstream sections of Hale Creek, which currently do not have full one-percent capacity. The sections are:

- Mountain View Avenue Bridge (i.e. box culvert)
- Channel reach from the confluence with Permanente Creek to Mountain View Avenue
- Channel reach from Mountain View Avenue to H-Line Station 9+60

The proposed work would involve replacing the existing trapezoidal channel with a deeper U-frame channel and replacing the single cell culvert at Mountain View Avenue with a double cell culvert. The existing concrete trapezoidal channels would be removed and replaced with a new concrete U-frame channel to meet the required freeboard above the 1-percent flood elevation (approximately 8 feet deep). To provide access to the channel near Mountain View Avenue, the U-frame channel will be widened to accommodate an access ramp and removable concrete separation wall.

New Diversion Structure

The existing diversion structure (at the upstream end of the Permanente Diversion) does not function correctly at the present time. The low flow intake was built into the invert of the channel and tends to silt up very quickly during winter flows. The diversion was later modified into an overflow weir parallel to the flow path; but that structure does not allow for low flows to continue down the natural Permanente Creek (Reach P7). Thus, the current flow profile at this location is that all flows, low and high, continue to Stevens Creek.

The new diversion structure will be a series of three screened 18-inch diameter pipes built along the invert of the trapezoidal channel. These new pipes will connect the main channel of Permanente Creek to a vault structure that conveys water into the existing 54-inch pipe. The 54-inch pipe then drains into an earthen trapezoidal channel downstream of Eastwood Drive. Upstream and downstream of the diversion structure, new concrete lining will be constructed to conform to the existing concrete trapezoidal channel. The existing maintenance ramp off Miramonte Avenue provides adequate maintenance access down to the invert of the channel and will not require any improvements.

Elements Not Included in the In-House Project

A portion of the geotechnical investigation work will be done by an outside consulting firm. The elements not included in our geotechnical investigation are:

- Rancho San Antonio Detention Basin
- Cuesta Park Annex Detention Basin
- Cuesta Park Inlet Channel
- McKelvey Park Detention Basin
- Blach School Detention Basin

The in-house project is currently in the design phase; therefore, the recommended project has been developed at this time. The project's current tasks, as stated in CPSGG's proposal, include the following:

- Investigate and evaluate the subsurface soil and groundwater conditions,
- Recommend design criteria for new floodwalls,
- Recommend lateral earth pressure and drainage design for new retaining walls/U-frame channels,
- Recommend design parameters for the new box culvert (bearing capacity and lateral earth pressure),
- Recommend design parameters for the new levee fill,
- Develop recommendations for site earthwork including fills and backfills,
- Develop recommendations for shoring and underpinning, as appropriate, and
- Provide seismic design criteria and other as-needed geotechnical information to support project design.

1.2 Scope of Services

As outlined in our proposal dated March 4, 2009, the purpose of our investigation was to characterize the subsurface conditions and geotechnical soil properties along the project site, and provide geotechnical design criteria for the proposed project.

The scope of our geotechnical investigation was limited to evaluating the general subsurface soil conditions and providing geotechnical recommendations for design and cost estimating purposes.

Based on the results of our investigations and engineering analyses, this report presents our design level conclusions and recommendations regarding the following:

- Site geology and seismicity
- Soil and groundwater conditions encountered
- Liquefaction potential
- Allowable bearing pressures for shallow foundations beneath floodwalls, box culvert, and retaining walls
- Sliding resistance for floodwalls and retaining walls
- Lateral earth pressures for floodwalls and retaining walls
- Design criteria for culvert replacement
- Slope stability analyses of new levees and floodwalls

- Levee settlement due to floodwall and levee fill loads
- Evaluation of levee seepage and erosion piping
- Seismic design criteria
- Evaluation of potential geological hazards, and
- Other construction considerations, as appropriate.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

District geotechnical staff explored subsurface conditions at the site by drilling thirteen (13) geotechnical test borings and advancing seven (7) Cone Penetration Tests (CPTs) along Permanente and Hale Creeks. We originally intended to drill fourteen borings; however, boring TB-8 could not be completed due to overhead trees obstructing the drill rig mast. Downstream of Highway 101, we also utilized the subsurface information from geotechnical borings drilled by others in June 2000[1]. Additionally, previous Boring B-4 drilled by District geotechnical staff in January 2003 will be utilized for flood control elements along Hale Creek. More detailed information for Boring B-4 is provided in Appendix F. Six of our thirteen borings were drilled using a truck-mounted Mobile B-53 drill rig equipped with 7 to 8 inch hollow stem augers. Seven of our thirteen borings required a specialty drill rig to facilitate overhead access issues (i.e. overhead power lines and tress). For these special circumstances, we used the Marl M-57 track-mounted drill rig equipped with a 6- ¼ inch hollow stem auger. In general, the hollow stem auger borings extended to a maximum depth of 45 feet below existing ground surface (bgs). The CPTs were advanced with a 30-ton truck mounted rig to maximum depths of 75 feet bgs. The shallowest CPTs, CPT-4 and CPT-7, were advanced to 45 feet bgs. All CPTs and borings were drilled from existing ground surface of the levee crest, channel maintenance road, or city streets. More detailed information of the CPTs is provided in Appendix A of this report.

Approximate locations of the borings and CPTs are shown on the Soil Boring Site Plan, Plates 2A through 2K. Stationing for the site plans along Permanente Creek begins downstream of Amphitheatre Parkway (C-Line Station 103+00) and ends at the diversion structure (C-Line Station 349+00). Stationing for the site plans along Hale Creek begins at the confluence with Permanente Creek (H-Line Station 0+00) and ends at Arroyo Road (H-Line Station 13+00) The site plan shows all the borings and CPTs drilled for this investigation and previously drilled locations by the District and others. The District's Survey unit surveyed the boring locations using the California State Plane Coordinate system; the horizontal datum is North America Datum (NAD) 1983 Zone 3, and the vertical datum is North America Vertical Datum (NAVD) 1988. The surveying coordinates for our borings and CPTs are presented in the following table:

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[1] Parikh Consultants, Inc., June 2000. "Geotechnical Investigation Report, Permanente Creek Levee Evaluation, Santa Clara County, California."

Geotechnical Boring Locations

Boring Name	Date Drilled	Easting	Northing
TB-1	06/23/2009	6,101,104.02	1,980,681.77
TB-2	06/24/2009	6,100,892.21	1,979,802.87
TB-3	07/02/2009	6,099,273.96	1,967,836.41
TB-4	06/30/2009	6,099,425.63	1,967,470.93
TB-5	06/29/2009	6,099,677.09	1,967,083.61
TB-6	07/02/2009	6,099,884.00	1,966,871.40
TB-7	06/25/2009	6,099,747.62	1,966,062.18
TB-9	06/29/2009	6,100,525.84	1,958,522.57
TB-10	07/01/2009	6,100,498.60	1,958,258.36
TB-11	06/25/2009	6,100,491.66	1,958,041.65
TB-12	06/24/2009	6,099,455.91	1,965,712.75
TB-13	06/30/2009	6,098,775.14	1,965,492.62
TB-14	07/01/2009	6,100,634.76	1,958,393.68

CPT Locations

Boring Name	Date Drilled	Easting	Northing
CPT-1	06/23/2009	6,100,932.57	1,980,670.06
CPT-2	06/23/2009	6,100,970.87	1,979,778.43
CPT-3	06/23/2009	6,100,855.81	1,979,000.89
CPT-4	06/23/2009	6,100,908.74	1,978,373.90
CPT-5	06/24/2009	6,099,652.64	1,967,053.57
CPT-6	06/24/2009	6,099,722.60	1,967,041.96
CPT-7	06/24/2009	6,100,477.91	1,958,352.57

Our field engineer logged the borings and obtained disturbed and relatively undisturbed soil samples at selected depths for visual classification and laboratory testing. In general, we obtained soil samples at depth intervals of approximately 1, 3 and 5 feet and at 5-foot intervals, thereafter, or at changes in subsurface lithology. We typically obtained soil samples using the following sampler types: a 2-inch outside diameter (O.D.), 1.4 –inch inside diameter (I.D.) Standard Penetration Test (SPT) sampler; a 3-inch O.D., 2.5-inch I.D. Modified California sampler; and a 3-inch O.D., 2.8-inch I.D. Shelby tube. Additionally, our field engineer obtained some samples at Boring TB-1 using a California sampler (2.5-inch O.D. with 2.0-inch I.D.)

Our drilling contractor drove the drive samplers with a 140-pound weight falling 30 inches for a penetration depth of 18 inches. Our engineer recorded the blow counts during driving of the sampler. For "non-standard" samplers, we converted[2] the recorded blow counts to equivalent SPT blow counts (N-values[3]), using appropriate correlations and then presented the equivalent SPT blow counts in our boring logs.

Detailed descriptions of the materials encountered for this investigation, with corresponding N-values, are presented on the Log of Borings, Plates 3 through 15. The soils are classified in accordance with the Unified Soil Classification System (ASTM D2487) shown on Plate 16[4].

Upon completion of drilling, our drilling contractor backfilled the geotechnical borings and CPTs with cement grout. At Borings TB-1, TB-7, and TB-9, our drilling contractor installed standpipe piezometers to measure future groundwater levels. The contractor backfilled each piezometer with No. 3 Monterey sand, hydrated bentonite pellets, cement grout, and capped each piezometer with a flush-mounted utility box set in place with concrete cement. Each standpipe piezometer consists of 2-inch diameter solid PVC pipe and screened PVC pipe with 0.02-inch-wide slot openings. A summary of the piezometer construction is presented below:

Piezometer	Cement Grout	Bentonite	No. 3 Sand	Solid PVC	Screened PVC
TB-1	0' – 16'	16' - 18'	18' – 45'	0' – 20'	20' – 45'
TB-7	0' – 26'	26' - 28'	28' – 45'	0' – 30'	30' – 45'
TB-9	0' – 31'	31' - 33'	33' – 45'	0' – 35'	35' – 45'

2.2 Geotechnical Laboratory Testing

We reexamined the samples in the laboratory to confirm our field classifications and to select representative samples for testing. Our laboratory testing program consisted of determining moisture content, dry density, grain size distribution, Atterberg limits, moisture-density relationship, consolidation, Unconfined Compressive strength (UC), Unconsolidated Undrained (UU) triaxial shear strength, and soil corrosion tests. Our laboratory testing program was performed by Cooper Testing Laboratories, a United States Army Corps of Engineers certified laboratory. The laboratory test results are either summarized on the logs of borings with the data presented in the manner described in the Test Data Key (Plate 16) or presented in detail in Appendix B.

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[2] Our conversion from California Modified blow counts to Standard Penetration N-values consists of multiplying the California Modified data by 0.70. This conversion is an approximation, and the "converted" N-values should be understood as such.

[3] Standard penetration N-values are defined as the number of blows of a 140-pound hammer falling 30 inches that are required to advance a standard sampler (2 inches O.D. and 1.4 inches I.D.) the final 12 inches of an 18-inch drive. The standard hammer driving mechanism utilizes a cathead-drum and a rope and pulley system.

[4] Changes in soil layers presented in our boring logs are transitioned with a solid line between soil symbols. Transition lines that are angled indicates the approximate range over which the change is likely to occur. Because soil changes often occur in between sampling intervals, our field engineer must estimate the depth in the soil transition. Steeper angled lines indicate greater uncertainty in estimating the depth range of the soil transition whereas a near horizontal line indicates that the transition was observed in the sample.

3.0 GEOLOGY AND SEISMICITY

The site is located near the southern end of San Francisco Bay within the Coast Ranges Geomorphic Province of Northern California. This province is characterized by a series of generally northwest-trending faults and folds. The geology of the area is strongly influenced by the San Andreas Fault System, which consists of a number of northwest-trending, right-lateral faults.

In general, the Santa Clara Valley basin consists of Cenozoic nonmarine (continental) sedimentary rocks and alluvial deposits. According to a geologic map compiled by Wagner, Bortugno, and McJunkin[5] in 1991, the area has been mapped as Alluvium comprising unconsolidated stream and basin deposits, clay to boulder size, and Older Alluvium comprising dissected alluvial deposits. Generally, Alluvium is mapped northeast of Central Expressway and Older Alluvium is mapped southwest of Central Expressway.

The San Francisco Bay region is recognized as being seismically active. Principal active faults in the Santa Clara Valley include the San Andreas, Hayward, and San Gregorio faults. Secondary faults that influence this region include the Calaveras, Monte Vista–Shannon, Sargent, and Zayante-Vergeles faults. Other faults mapped in this region include the Evergreen and Silver Creek faults. The faulting map compiled by Bortugno, McJunkin, and Wagner[6] in 1991, also shows an unnamed fault with evidence of Quaternary (last 2,000,000 years) displacement as crossing the project; however, only faults with evidence of more recent Holocene (last 10,000 years) displacement are generally considered active. Earthquakes occurring along these and other faults are capable of generating strong ground shaking at the site. The faults capable of producing the strongest ground shaking at the site are the Monte Vista-Shannon and the San Andreas. The relative distance to the nearest principal and secondary faults are presented below:

Fault Name	Direction from Site	Distance (miles)	Distance (kilometers)	Fault Type
Monte Vista-Shannon	Southwest	2.2 to 5.3	3.5 to 8.5	Secondary
San Andreas	Southwest	5.3 to 7.9	8.5 to 12.7	Principal
Hayward (southeast extension)	Northeast	10.9 to 12.7	17.5 to 20.4	Principal
Hayward	Northeast	11.7 to 14.8	18.9 to 23.8	Principal
Calaveras	Northeast	15.1 to 16.4	24.3 to 26.4	Secondary

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[5] Wagner, D.L., Bortugno, E.J., and McJunkin, R.D., published 1991. "Geologic Map of the San Francisco-San Jose Quadrangle, California, 1:250,000." State of California, Division of Mines and Geology, Regional Map Series.

[6] Bortugno, E.J., McJunkin, R.D., and Wagner, D.L., published 1991. "Map Showing Regency of Faulting, San Francisco-San Jose Quadrangle, California, 1:250,000." State of California, Division of Mines and Geology, Regional Map Series.

San Gregorio	Southwest	17.5 to 19	28.1 to 30.5	Principal
Sargent	Southeast	17.4 to 21.2	28 to 34.1	Secondary

As shown in the above table, the site is approximately 2 to 5 miles from a Secondary Seismic Source (the Monte Vista-Shannon Fault) and 5 to 8 miles from a Principal Seismic Source (San Andreas Fault). The site is not within a State of California Alquist-Priolo Earthquake Hazard Zone.

4.0 SURFACE AND SUBSURFACE CONDITIONS

4.1 Surface Conditions

The project consists of upgrading approximately 1.2 miles of the existing channelized creek. The reaches of Permanente and Hale Creeks flow through developed regions of Los Altos and Mountain View consisting of mixed commercial, residential, and industrial areas.

Between Highway 101 and the San Francisco Bay, the creek is contained within a combination of earthen channels and levees. Between the confluence of Hale and Permanente Creeks and Highway 101, the creek is contained within rectangular and trapezoidal concrete-lined channels and floodwalls. Between Arroyo Road and the confluence of Permanente and Hale Creeks, the creek is contained within trapezoidal concrete-lined channels. The Permanente Diversion Channel is a concrete lined trapezoidal open channel.

Based on our review of project cross-sections, from the beginning of the project located downstream of Amphitheatre Parkway to Hwy 101, the existing earthen channel and levee bank heights (measured from the bottom of the creek) range from about 7 feet to 16 feet. The creek channel and levee side slopes are inclined at about 1:¾ horizontal to vertical (H:V) near Hwy 101 and about 1:2½ (H:V) downstream of Amphitheatre Parkway. Channel side slopes immediately downstream of Hwy 101 are concrete lined. Currently, the channel bank and levee maintenance roads are about 8 feet to 13 feet wide and constructed from aggregate base course materials and asphalt concrete. At the time of drilling, we observed grass, shrubs, weeds, and trees along the levee and channel slopes.

4.2 Subsurface Conditions

The purpose of this field investigation was to evaluate soil conditions at the site in support of the design phase for this project. Accordingly, the borings and CPTs drilled for this report were spaced within 500 feet of new or previous borings. The subsurface conditions described below are based on these test borings.

Existing Levees and Creek Banks Downstream of Hwy 101

Our borings and CPTs along this reach included TB-1, TB-2, and CPT-1 through CPT-4. In general, the existing levee fills and creek banks located downstream of Hwy 101 consists of sandy lean clay, clayey sand, and occasional thin layers of sand. Thin layers of sand within the upper 8 feet, about ½ to 1 foot thick, were encountered while advancing CPT-2 and CPT-3. The fill bank material encountered in borings were medium stiff to very stiff and appear to be well compacted. Previous borings conducted by Parihk Consultants (June 2000) identified layers of fat clay with traces of sand and gravel in the levee fill at borings B-1, B-2, B-3, and B-4.

Below the levee fills and creek bank soils, we encountered varying layers of sand, silt and lean clay down to the maximum depth of the borings. The silt and clay layers contain varying amounts of fine to medium sand and are generally stiff to very stiff. The sand deposits were typically water bearing layers with trace amounts of fines and gravel and characterized as medium dense to very dense. Occasional loose sub layers of sand were encountered immediately below groundwater. These soils are indicative of the alluvial deposits that have been mapped in this area. Due to the extent of the project and the nature of our borings, the subsurface strata encountered appear quite varied and non-continuous.

Existing U-Frame Channel along Permanente Creek

Our borings and CPTs along this reach included TB-3 through TB-7, and CPT-5 through CPT-6. The soils encountered in our borings and CPTs adjacent to the U-frame walls and culvert consisted mostly of clayey sand and sandy lean clay. Occasionally, we encountered layers of sand (with little or no fines), silt, and fat clay. The sand and clay layers were highly variable in borings TB-3 through TB-5. However, in borings TB-6 and TB-7, the upper 15 feet were mostly sandy lean clays. In general, the sandy lean clays and clayey sands are characterized as being stiff to very stiff, and medium dense to very dense. Occasional loose layers of sand were encountered either close to the surface or immediately below groundwater.

Hale Creek

Our borings along this reach included borings B-4 (2005 Planning Study), TB-12 and TB-13. The soils encountered in our borings along this reach consisted mostly of course grained sands and gravels with silt or clay. Occasional layers of lean clay interbedded between the sand layers were also encountered. The sand, gravel, and lean clay layers can be generally characterized as medium dense to dense, and stiff to hard. Groundwater was not encountered in any of these borings.

Permanente Diversion Structure

Our borings along this section of the project included borings TB-9 through TB-11, TB-14, and CPT-7. The soils encountered near the diversion structure consisted mostly of course grained sands with varying amounts of silts or clays intermixed. Layers of sandy lean clay, sandy silt and silty gravel were also observed, but much less than other reaches further downstream. The sand layers can be generally characterized as medium dense to dense. In boring TB-11, we did encounter some very loose fill material in the upper 5 feet that appears to have been poorly compacted. Groundwater was not encountered in any of these borings.

More specific details on the soils encountered in each boring and CPTs are shown on the Boring Logs (Plates 3 through 15) and Appendices A, F and G.

5.0 GROUNDWATER CONDITIONS

We measured the depth to groundwater during and after drilling our field investigation. Additionally, we installed three piezometers for future groundwater measurement. A summary of the groundwater readings encountered in the piezometers is presented in the following table. Groundwater readings measured during drilling indicate the first encounter of free water on the auger or sampler. Thereafter, our field engineer returned to each piezometer and took subsequent groundwater readings to monitor groundwater fluctuation. The groundwater readings are presented in the table below:

		Groundwater Readings							
Boring Name	Ground Elevation (feet)*	During Drilling		10/20/09		1/28/10		3/12/10	
		Depth (feet)	Elev. (feet)	Depth (feet)	Elev. (feet)	Depth (feet)	Elev. (feet)	Depth (feet)	Elev. (feet)
TB-1	15.1	15.9	-0.8	15.7	-0.6	14	1.1	15.2	-0.1
TB-7	113.1	NE**	NE	NE	NE	NE	NE	NE	NE
TB-9	186.9	NE	NE	NE	NE	NE	NE	NE	NE

* Elevations and coordinates were surveyed by the District's Surveying unit and refer to NAVD 1988 and NAD83, respectively.
** NE = groundwater not encountered

We also present a summary of groundwater readings including borings performed in 2009 for this investigation, the District's investigation for the 2005 planning study, and Parihk's 2000 investigation. This summary is presented in Appendix C.

It should be noted that fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, channel water levels, and other factors.

6.0 ENGINEERING ANALYSIS

6.1 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers located close to the ground surface. These soils lose strength during cyclic loading, such as that imposed by earthquakes. During the loss of strength, the soil acquires "mobility" sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie relatively close to the ground surface.

In general, the medium stiff to stiff lean clay encountered in our borings is not susceptible to liquefaction or densification. However, we encountered occasional layers of sandy soils that are potentially liquefiable in our borings and CPTs drilled along Permanente Creek in the areas downstream of Hwy 101 (C-Line Station 131+32) and between Park Drive and Mountain View Avenue. Upstream of Mountain View Avenue along Permanente and Hale Creeks and to the extent of our investigation near Miramonte Avenue (C-Line Station 348+00), the soils encountered in our borings and CPTs were either clayey or relatively dense and not susceptible to liquefaction.

The District's engineering geologist performed a deterministic seismic analysis of peak ground acceleration from known faults near the project site. Using the attenuation relationships by Abrahamson and Silva (1997), he estimated that a maximum credible earthquake, with a moment magnitude (M_w) of 7.9, occurring along the San Andreas fault (1906 segment) would result in a peak bedrock acceleration of 0.36g (expressed as a fraction of the acceleration due to gravity, g) at the project site. We also reviewed the California Geological Survey's (CGS) website of probabilistic seismic hazards assessment of peak ground acceleration with a 10 percent probability of being exceeded in 50 years. Based on the CGS hazard maps, we estimate the peak bedrock acceleration (for soft rock) to be 0.46g. The corresponding peak ground acceleration (at the ground surface) was estimated to be 0.48g. We conservatively used the higher CGS value in our liquefaction analysis.

We evaluated the liquefaction potential of the near surface sand layers encountered in our borings. Based on studies presented by Cetin et al.[7] and Moss et al.[8], we calculated the cyclic shear resistance and the cyclic shear stress using field SPT and CPT data. The factor of safety against liquefaction was then calculated as the ratio of cyclic shear resistance to cyclic shear stress.

The results of our analyses indicate that sand layers in Borings TB-1, TB-3, TB-4, CPT-1, CPT-2, and CPT-3 have factors of safety against liquefaction of less than 1.0. In theory, factors of safety less than 1.0 suggests that the sand deposit has the potential to liquefy during a major earthquake. For estimates of settlement due to liquefaction, refer to Section 6.4 "Settlement."

6.2 Slope Stability

As currently planned, the improvements downstream of Amphitheatre Parkway will consist of raising the western levee. In evaluating the reach downstream of Amphitheatre Parkway, we utilized boring logs from a report prepared by Parikh Consultants to supplement our own subsurface data. For completeness, we have included Parikh's report in Appendix F.

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[7] Cetin, K. Onder et al. December 2004. "Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction." Journal of Geotechnical and Geoenvironmental Engineering. Pgs 1314-1340.

[8] Moss, R.E.S. et al. August 2006. "CPT-Based Probabilistic and Deterministic Assessment of In Situ Seismic Soil Liquefaction Potential." Journal of Geotechnical and Geoenvironmental Engineering. Pgs 1032-1051.

Based on the 60% Design Plans, we understand the new height of the levee will be raised by as much as 2½ feet above the existing west bank. The new levee will be constructed as a landside enlargement beginning at WB-1 Line Station 0+00 and ending at Station 5+05.

In accordance with the US Army Corps of Engineer's (Corps) Manual EM 1110-2-1913, the critical cases and corresponding minimum factor of safety requirements are presented in Table 1 below. Our slope stability analyses were performed using Spencer's method of slices and the computer programs Slope/W and UTEXAS 4. Case Nos. I, II and IV were performed using Slope/W and Case No. III was performed using UTEXAS 4. The side slope inclination of the levee in our analysis is 2:1 (H:V). A summary of our slope stability analyses for the new levee is presented in the Table 2 below.

TABLE 1

Case No.	Design Condition	Slope Analyzed	Minimum Factor of Safety
I	End of Construction	Creekside and Landside	1.3
II	Steady seepage from full flood stage	Landside	1.4
III	Sudden Drawdown	Creekside	1.1
IV	Earthquake	Creekside and Landside	1.1

TABLE 2

	New Levee Factor of Safety ➔ With 2.5 feet of Additional Fill			
Location	Case I	II	III	IV
Station 105+45	5.1 (CS) 6.4 (LS)	1.8 (LS)	1.2 (CS) 1.6 (LS)	1.7 (CS) 2.0 (LS)

Note: (CS) Creekside slope (LS) Landside Slope

Several floodwalls along Permanente Creek will also be used to retain maintenance roadway fill. Portions of the EB-2 line and WB-3 line floodwalls will be filled about 2.5 feet to 3 feet to raise the existing levee maintenance road. The design plans show the retaining walls are about 10 to 12 feet apart from top of wall. The critical condition is the additional loadings from the retaining wall and fill and whether the creek bank has adequate stability. We performed a slope stability analysis of the WB-3 floodwall and fill configuration (C-Line Station 118+50). This configuration included a Type II outer floodwall with a uniform load equal to 1,510 pounds per square foot (psf) and a Type IV inner floodwall with a uniform load equal to 640 psf. Uniform loading of the floodwalls were provided to us by the project structural engineer. The new maintenance road is approximately 2.5 feet above the existing levee road surface. The results of the slope stability analyses for this floodwall indicate the factor of safety against slope failure is between 2.4 and 5. The results of our slope stability analyses are presented in Appendix D.

6.3 Seepage

We evaluated the potential for seepage to emerge from the slope face and through the foundation soils of the proposed levee by analyzing one cross-section at C-Line Station 105+45 (located downstream of Amphitheatre Parkway) using the computer program SEEP/W version 5. We performed a steady state seepage analysis to estimate the hydraulic gradient at the landside toe of the levee and underlying soil layers near the surface. Under sustained flood levels, our analysis indicates that the wetting front exits from the levee's landside slope face. Upward seepage would not likely occur along the landside toe due to the presence of the lean clay upper stratum. The concentration of seepage lies below the upper lean clay and into the more pervious sand layer. Under steady state seepage conditions, we determined the hydraulic gradient from seepage at the toe of the landside slope is about 0.55. Typically when the exit gradient at the landside is between 0.5 and 0.8, the Corps recommends constructing a berm, with a minimum thickness of 5 feet, along the landside toe to control seepage and potential heaving of the upper clay layer. We should note that the 100-year flood hydrograph duration is about 36 hours with peak flows lasting about 1 hour. In our experience, the time required to develop the steady state condition would be much longer. Historically, the levees downstream of Highway 101 have not experienced seepage problems, heaving, or boiling along the landside toe of our levee slopes. Therefore, we judge the development of steady state condition to be an unlikely event. The results of our seepage analyses and the 100-year flood hydrograph are presented in Appendix E.

6.4 Settlement

As currently planned, the western levee downstream of Amphitheatre Parkway will be raised by placing additional fill to provide the required freeboard. According to the 60% Design Plans, the fill height will range from 2 feet to 2 ½ feet at the crest of the new levee and about 6 feet of new fill over the landside toe. As a result, additional loads will be imposed on the underlying clay soils and ground settlement will occur. We estimated the coefficient of virgin compression, coefficient of recompression, and the preconsolidation pressure from the laboratory test data of underlying clay soils. We estimate that the total settlement induced by raising the levee will be up to 3½ inches. Differential settlement should be approximately half of the total settlement.

Between Amphitheatre Parkway and Highway 101, immediately downstream of Mountain View Ave (along Permanente Creek), and immediately downstream of Arroyo Road (along Hale Creek), new floodwalls will be constructed over the existing levees and creek bank to provide the required freeboard. Sustained dead loads to the floodwalls will result in settlement of the existing levees and creek bank. We estimate that the total settlement from loading by the floodwalls will be up to 2 inches. Differential settlement should be approximately half of the total settlement.

At the intersection of Mountain View Avenue and Hale Creek, a new double cell box culvert will replace the existing single cell box culvert to provide greater flood capacity. Based on our discussions with the project structural engineer, we understand that the culvert can withstand up to ½ inch of total settlement. Since settlement governs bearing capacity under this condition, we have provided allowable bearing capacity values in Section 8.3 'Foundation Design' such that the estimated total settlement will be ½ inch or less.

Settlement may also result from liquefaction of the loose sand layers encountered in our borings. The following table summarizes settlement induced by liquefaction in our borings.

Boring	C-Line Station	Depth of Sand Layer(s) (feet)	Depth to Groundwater (feet)	Settlement (inches)
TB-1	107+20	11–14.5 19 - 26	10.5*	3
TB-3	241+45	37–42 42–44	37*	2
TB-4	245+30	36–42	36*	1.5
CPT-1	107+55	68–70	16.5	0.5
CPT-2	116+55	51–54	18.5*	0.5
CPT-3	124+35	39–40 42–43	20.5*	1

* Estimated high groundwater depth

Based on the available subsurface data, we estimate that up to 3 inches of ground settlement may occur at Boring TB-1 and 2 inches or less at Borings TB-3, TB-4 and CPT-1, CPT-2, and

CPT-3 due to seismically induced liquefaction. Borings TB-1, CPT-1, CPT-2, and CPT-3 are located along the levee and creek banks along Permanente Creek and downstream of Highway 101. Borings TB-3 and TB-4 are located along Permanente Creek between Park Drive and Mountain View Avenue. This settlement is in addition to the static load induced total settlements given for the floodwalls and levee. Studies by Iwasaki et al. (1982) have shown that the contribution of settlement of soil layers diminishes as the depth of the layer increases. The threshold depth at which settlement of an individual layer cannot be traced at the ground surface was found to be 60 feet. Accordingly, we judge that liquefaction in the sand layer of CPT-1 will not produce settlement at the ground surface of the proposed levee. The other borings drilled for this investigation did not encounter liquefiable soils.

7.0 DISCUSSIONS AND CONCLUSIONS

This report is based on test boring data that is intended for a design-level study. To aid the project team, we have provided geotechnical criteria for design purposes. Based on the available subsurface information to date, we conclude that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations presented in this report are incorporated into the project design plans and specifications. The main geotechnical considerations for the project are discussed in the following sections.

7.1 Liquefaction and Other Seismic Considerations

The site is located in a seismically active region of California. Significant earthquakes in the Bay Area have been associated with movements along well-defined fault zones. Earthquakes occurring along the San Andreas, Monte Vista-Shannon, Hayward or any of a number of other Bay Area faults have the potential to produce strong ground shaking at the site. For this reason, the structures and embankment slopes should be designed to resist or accommodate lateral and uplift forces generated by earthquake shaking, in accordance with local design practice.

Settlement can occur at some sites as a result of seismic ground shaking due to liquefaction or densification of the subsurface soils. As presented in Section 6.1, our engineering analyses indicate that the soils encountered in our borings downstream of Highway 101 and between Park Drive and Mountain View Avenue are potentially liquefiable. A potential effect of liquefaction is ground settlement. Based on available subsurface data, we estimate that up to 3 inches of liquefaction induced settlement may occur following a major earthquake. Liquefaction induced settlement is in addition to the static load induced settlement previously summarized in Section 6.4 and should be considered in the design.

Other potential effects of liquefaction are lateral spreading and bearing capacity failure. Lateral spreading is a horizontal movement of soil which can occur during an earthquake. Typically, lateral spreading occurs on sites underlain by soft or liquefiable soils adjacent to a free face (i.e. a river bank, coastal bank, or embankment). Bearing capacity failure is the soils inability to support the levee or floodwall loads due to a lack of soil strength. Generally, if a liquefiable soil layer is located within the depth equal to the width of the footing or levee and the nonliquefiable soil layers do not have sufficient strength, bearing capacity failure can occur. Because portions of the project are adjacent to a free face, there is a potential for lateral spreading. However, according to our subsurface data and existing creek bank profiles, we judge the potential is low since the liquefiable sand layers are deeper than the free face of the levee embankments and creek banks. The potential for bearing capacity failure due to liquefaction along the levee and floodwall areas of Permanente and Hale Creek is also low because the liquefiable sand layers encountered in our borings are deeper than the width of the levee and floodwall footings.

Other geologic hazards such as lurching or fault rupture are considered to be unlikely at this site due to the nature of the near surface soils, topography, and the distance of the site from known active faults.

7.2 Slope Stability

Based on the results of the slope stability analyses, the proposed levee raising downstream of Amphitheatre Parkway with side slope inclinations of 2:1 (H:V) has adequate stability under static and seismic loading conditions. Our analyses assume that the levee fill will meet the strength requirements provided in Section 8.1.4 of this report. If the contractor cannot meet the levee fill strength requirements, the project design team should advise us so we may evaluate alternatives and provide further recommendations.

The results of the slope stability analyses for floodwalls with maintenance roadway fill indicate the factor of safety against slope failure is between 2.4 and 5. The minimum factor of safety for this condition is 2. Based on the results of the slope stability analysis, the floodwalls have adequate stability under static loading conditions.

The results of our slope stability analyses are also presented in Appendix D.

7.3 Channel Bank Erosion

In general, soil banks adjacent to flow velocities greater than 6 feet per second should be protected from erosion. Depending on the site conditions and the severity of erosion potential, erosion may range from hydro-seeding with deep rooted vegetation to engineered rock slope protection. Erosion protection should be implemented into the project design plans along creek banks where new floodwalls and levees will be constructed to protect the toe of slopes from scour and potential instability.

7.4 Seepage

Our subsurface investigation encountered mostly sandy clays, clayey sands, and occasional thin layers of sand in the existing levee and creek banks located downstream of Highway 101. While the sandy clays and clayey sands are relatively impermeable, the thin sand layers are permeable and susceptible to subsurface seepage. Sand pockets within the levee could be a potential pathway for the initiation of erosion piping. Erosion piping is the result of water seepage that emerges from the levee face or toe that, if the exit gradient is high enough, starts a process of erosion and forms a tunnel-shaped passage (or pipe) within or beneath the levee. When the passage slowly forms its way back to the water source (i.e. creek flows), a mixture of soil and water rushes through the passage, undermining the levee or floodwall, and flooding the area that was protected by the structure.[9] If sand layers or pockets are encountered during excavation for the floodwall footings and levee, the sandy material should be over excavated and replaced with compacted levee fill as discussed in Section 8.1.6.

We performed a steady state seepage analysis downstream of Amphitheatre Parkway to evaluate potential through and under seepage failures along the landside toe of the proposed levee. Because the 100-year flood event duration is very short, about 36 hours with peak flows occurring over 1 hour, we judged the potential for the development of steady state seepage to be low. However, as a preventative measure we recommend constructing a minimum 3 foot deep compacted clay cap along the landside toe of the new levee. This can be accomplished by extending the 3-foot deep keyway fill, as discussed later in Section 8.1.3, to the edge the District's proposed right-of-way. The compacted clay cap will reduce the potential for boiling and heaving along the landside toe area. We also recommend the project team include regular monitoring of the new levee during flood events in the Watershed's Operations and Maintenance Plan.

7.5 Settlement

Ground settlement resulting from additional fill loads, floodwall loads, and soil liquefaction will induce total and differential settlement below the levee, floodwalls, and culvert. Estimates of settlement were provided in Sections 6.4 and 7.1. During preparation of design plans and specifications, the design team should consider overbuilding levees to accommodate settlement and the effect of settlement on new culverts and floodwalls. For further recommendations regarding settlement of foundations, see Section 8.3.1, "Foundation Preparation" of this report.

7.6 Shoring, Underpinning and Dewatering

Based on the 60% Design Plans, the following structures will require deep excavations: 1) a new underground box culvert will be constructed beneath Mountain View Avenue at Hale Creek, and 2) U-frame channel excavation along Permanente Creek. Excavations for the box culvert will be within existing streets. Shoring will likely be required to support the sides of the excavation and the adjacent street. We understand the project team plans to excavate the existing invert slab of the U-frame channel. Because the channel is adjacent to residential homes, underpinning will be required to support the existing U-frame walls.

In our field investigation, we did encounter groundwater ranging from 39 feet to 45 feet below ground surface in our test borings located along Permanente Creek between Park Drive and Mountain View Avenue. All test borings upstream of Mountain View Ave along Permanente Creek and along Hale Creek did not encounter groundwater within the upper 45 feet. Because the excavations for the underground structures will be within the upper 10 feet, dewatering the excavation due to groundwater will likely not be necessary. If the excavation is performed during the winter months, localized dewatering may be required. Dewatering can likely be accomplished from sumps within the excavation; however, relatively higher flows should be expected in localized sandier zones.

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[9] Naval Facilities Engineering Command (NAVFAC), September 1986. Soil Mechanics Design Manual 7.01, pgs 262 – 263.

The performance of shoring systems and site dewatering is highly dependent on the construction methods and procedures employed. The design, construction, and maintenance of all necessary shoring and dewatering systems, as well as the protection of existing site improvements and utilities, should be the responsibility of the contractor. Excavations that will be deeper than 5 feet and entered by workers should be shored or sloped for safety in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). For relatively narrow utility trenches exceeding 5 feet deep, an internally braced shoring system or use of temporary slopes may be appropriate. General geotechnical recommendations for apparent earth lateral loads for temporary shoring and underpinning are provided in Section 8.4 "Lateral Earth Pressures" of this report. The project geotechnical and structural engineers should review all proposed shoring systems.

7.7 Corrosivity Evaluation

We submitted three soil samples to Cooper Testing Laboratory for corrosivity analyses. All samples selected for corrosivity analyses are from our borings located near the U-frame channel or culvert locations. The following table summarizes the results of the laboratory testing.

Sample I.D.	pH	Resistivity (100% Saturation)	Chloride (mg/kg)	Sulfate (mg/kg)
TB-3 @ 5.5'	7.0	3,020	7	<5
TB-6 @ 11'	7.3	1,670	55	<5
TB-12 @ 11'	6.5	6,644	3	<5

We reviewed the California Department of Transportation (CalTrans) Bridge Design Specifications[10] to evaluate the corrosivity test results. CalTrans considers a site to be corrosive with sulfate concentrations greater than 2,000 mg/kg and/or a pH of less than 5.5. Since the corrosion results from our samples do not meet these criteria, the following recommendations are for a non-corrosive environment.

- Minimum concrete cover for protection of reinforcement is 3 inches,
- Minimum requirements for protection of reinforced concrete is to use Type I-P (MS) modified or Type II modified cement.

For structures within a marine atmosphere, which includes the atmosphere of land within 1,000 feet of ocean or tidal water, CalTrans requires a minimum concrete cover. Therefore, if any of the structures along the site fall within these limits, the structures should have at least 3 inches of concrete cover to protect against corrosive elements in the air. The recommended cover is for a 75-year design life.

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[10] California Dept. of Transportation, September 2003. Bridge Design Specifications. Section 8.22, pgs 8-37 to 8-39.

A summary of the corrosion testing is also presented in Appendix B.

7.8 Foundation Considerations

The existing levees and creek banks along portions of Permanente and Hale Creeks will be improved for flood protection purposes by constructing new floodwalls. The induced loads of the floodwalls will cause the ground to settle. We present recommendations to reduce the settlement of floodwalls in Section 8.3.1, "Foundation Preparation" of this report.

8.0 RECOMMENDATIONS

8.1 Grading and Earthwork

8.1.1 Clearing, Grubbing, and Stripping

The top of the existing levee, or the other areas to receive fill, should be cleared of existing vegetation and debris. Clearing should consist of the complete removal above the ground surface of all trees, stumps, downed timber, snags, brush, vegetation, loose stone, and other debris from within the construction limits.

The area within the levee foundation should be grubbed of all roots, wood, old paving, and other objectionable material. Tap roots and lateral roots over 2 inches in diameter should be grubbed to a minimum depth of 1½ feet below the ground line. All holes and depressions caused by grubbing operation should be flattened by backfilling and compacting to a density equal to the adjoining undisturbed material to avoid compressibility inconsistency of the foundation beneath the levee.

Borrow areas, mixing areas, and areas cleared for levee construction should be stripped of all organic topsoil and shallow root systems to an average depth of 6 inches. Stripped soils contain nutrients for plants and may be returned to the top of the graded surface in areas to be landscaped. Stripping is not considered suitable for use as fill material, but it may be placed in a layer no thicker than 6 inches on the finished slopes of the embankment, or in any thickness outside the completed engineered fill to favor revegetation.

8.1.2 Embankment Subgrade Preparation

Following clearing and stripping, all loose or soft soils lying at the sides, bottom, or top of levees should be removed to prepare a firm, uniform subgrade. After excavation of the levee embankment foundation, pits and other cavities or depressions should be backfilled and compacted. Immediately before placing fill material, the entire earth surface on or against which fill is to be placed should be scarified to a depth of six inches, moisture conditioned to about the optimum to three percent above the optimum moisture content, and compacted to at least 90 percent relative compaction, as obtained by the ASTM D1557 test procedure. If this surface, or other surface that is to receive fill, is compacted so that a plane of seepage or weakness might result, it should again be thoroughly scarified before placing material on it. The foundation should be aerated if wet or moistened if dry to achieve the proper moisture content. Following subgrade preparation, the ground surface should be kept moist to prevent excessive shrinkage due to moisture loss.

8.1.3 Keyway and Benching

According to the design plans, the new levee fill will be constructed as a landside enlargement. To support the new fill on native soils, we recommend constructing a keyway along the landside toe of the new levee embankment. The keyway should be at least three feet deep, and side slopes of the keyway excavation should be at least 1:1 (H:V) or flatter. As discussed in Section 7.4, the width of the compacted keyway should be extended to the District's proposed right-of-way to protect the levee toe area against the potential effects of under seepage. We also recommend excavating benches into the existing levee fill to provide a strong interface between the existing levee and new levee fills. Bench excavations should be at least 2 feet in height measured from a horizontal line projecting from the slope face. All keyway and benching excavations should be backfilled with levee embankment fill as described below in Section 8.1.6 "Fill Placement and Compaction."

8.1.4 Levee Embankment Fill Materials

In accordance with the District's *Levee Safety Technical Guidance Manual*, Final Report, dated April 29, 2002, levee fill material should be free of organic matter, deleterious substances, debris, and rocks or lumps. Levee fill material should meet the following grading requirements in accordance with the Particle Size Analysis Test ASTM D422: at least 100 percent of the fill materials should be finer than a ¾ inch U.S. Standard Sieve; at least 80 percent of the fill materials should be finer than a No. 4 U.S. Standard Sieve (4.76 mm); and at least 50 percent of the fill materials should be finer than a No. 200 U.S. Standard Sieve (0.074 mm). The clayey levee fill should be of low to medium plasticity as classified by the Unified Soil Classification System having a Plasticity Index (PI), in accordance with the Atterberg Limits Test ASTM D4318, between 10 and 20 and a liquid limit of less than 40. Material that is primarily silt should be avoided. Also, the levee fill materials should be relatively impervious (permeability after compaction should be less than 10 to the power minus 6 centimeters per second [10^{-6} cm/sec]) in accordance with the Falling Head Permeability Test ASTM D5084.

Levee fill shall also meet the following strength requirements:

Strength Parameter	Required Minimum Value	Testing Procedure
Undrained Shear Strength	2,000 psf	ASTM D2850
Effective Friction Angle	30 degrees	ASTM D4767

Effective Cohesion Intercept	100 psf	ASTM D4767
Total Friction Angle	25 degrees	ASTM D4767
Total Cohesion Intercept	200 psf	ASTM D4767

All test samples for the strength requirements in the above table should be compacted to at least 90% relative compaction and moisture conditioned to 0-3% above optimum as obtained from the Modified Proctor Compaction Test ASTM D1557. Results from testing procedure ASTM D4767 should be based on three remolded test samples consolidated at confining pressures of 250, 500, and 1000 psf. Results from testing procedure ASTM D2850 should be based on one remolded test sample at a confining pressure of 500 psf.

8.1.5 Structural Fill Materials

Structural fill material should generally be used outside of the levee embankment footprint and beneath pavements, sidewalks, or foundations. Structural fill may be used to support pavements on top of the levee provided the structural fill is limited to the levee freeboard. Structural fill should be free of organic matter, deleterious substances, debris, and rocks or lumps larger than 4 inches in greatest dimension. No more than 15 percent of the rocks or lumps should be larger than 2½ inches, and at least 20 percent of the fill materials should be finer than a No. 200 U.S. Standard Sieve (0.074 mm). Structural fill should have a low expansion potential. A low expansion potential for structural fill should be defined by a liquid limit of less than 40, and a plasticity index (PI) between 8 and 15. Structural fill should be checked, and if necessary, tested prior to placement and compaction following the same ASTM standard procedures described above for levee fill.

8.1.6 Fill Placement and Compaction

Materials placed on the levees should be placed or spread in layers not more than eight inches in uncompacted thickness on the levee prior to compaction. Each layer should be spread evenly and thoroughly mixed during spreading to ensure uniformity of material in the fill and eliminate the potential placement of continuous pervious layers. Mixing should consist of disking each lift with a minimum of two passes before compaction.

Scour hole rock fill should be placed in layers no more than two feet thick. Layers should be, where practical, approximately horizontal and have sufficient crest or slope to drain during construction. The fill should be placed in parallel layers. When the surface of any compacted layer cannot bond properly with the next layer, it should be scarified before placing the next layer on it.

The soil must be at the proper moisture content before it is compacted. Soils that are too wet or too dry will not compact well. Sandy soils should be moisture conditioned to within two percent of the optimum moisture content (as obtained by ASTM D1557), and clayey soils should be moisture conditioned to between the optimum moisture content and three percent above the optimum moisture content.

If the material is too dry for proper compaction, it should be pre-wetted in the mixing or excavation area to uniformly distribute sufficient moisture in each layer before rolling to permit the desired compaction. Material that is too wet should be spread adjacent to the site and permitted to dry, assisted by scarifying if necessary, until the moisture content is reduced sufficiently to permit placement and compaction. The final adjustment of moisture should be made on the fill, as required, until a uniform distribution of moisture is obtained.

When the moisture content and conditions of the spread layers are satisfactory, each layer should be compacted to not less than 90 percent relative compaction, based on the maximum dry density obtained by ASTM D1557. The layer to be compacted should extend beyond the final grade lines and be trimmed back to ensure adequate compaction at the levee faces. Portions of the embankment that the roller cannot reach should be compacted by any approved method to 90 percent relative compaction. Structural backfill under pavements, sidewalks, or foundations should be compacted to at least 95 percent relative compaction.

Where sheepsfoot rollers are used, the soil may be disturbed to a depth of several inches. Density tests should be taken in compacted material below the disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below the required relative compaction, the particular layer or portion should be recompacted until the required density has been obtained.

8.1.7 Finished Grade, Grade Slope and Tolerance

The finished levee crest elevation should be overbuilt to compensate for post-construction settlement. Fill slopes should be compacted by means of sheepsfoot rollers or other suitable equipment. Compacting operations should be continued until the slope surfaces meet the specified compaction requirements. Generally, because of difficulty obtaining compaction at the edges, levee slopes should be overbuilt by a minimum of 12 inches, and subsequently trimmed off to the desired grade in order to achieve a firm, uniform slope surface.

After the slopes have been graded, they should be watered, track rolled, seeded, and mulched for erosion control. To reduce potential erosion, no slope should be left standing through a winter season without providing for erosion control.

Up to six inches of excess material above the prescribed grade is permitted in the final dressing if the excess material is so distributed that the crest of the levee drains and there are no abrupt humps or depressions in the surface. More than six inches of excess material may be allowed at locations where such modifications will not impair the design or appearance of the levee.

8.2 Seismic Design Criteria

Structures should be designed to resist lateral forces generated by earthquake shaking in accordance with local design practice. This section presents seismic design criteria for the 2009 International Building Code (IBC).

As defined in the 2009 IBC, the following criteria are appropriate for the site:

Site Class = D

Mapped Spectral Response Acceleration at 0.2-Second Period:	$S_S = 1.75g$
Mapped Spectral Response Acceleration at 1-Second Period:	$S_1 = 0.75g$

Refer to Section 1613 of the 2009 IBC for more information on seismic design.

The project extends approximately 5 miles (8 km) from downstream of Amphitheatre Parkway to Miramonte Avenue. The southern extent of the project (i.e. Miramonte Avenue) is approximately 2.2 miles (3.3 km) from the Monte Vista-Shannon Fault. The northern extent of the project (i.e. Amphitheatre Parkway) is approximately 5.3 miles (8.5) km from the Monte Vista-Shannon Fault.

In general, the site will behave like one having a stiff soil profile (S_D). Loose sand layers of potentially liquefiable soil were identified in our subsurface investigation. The impacts of liquefaction in terms of ground settlement were discussed in Section 7.1. From a seismic standpoint, the occurrence of liquefaction will tend to reduce ground surface motions. Therefore, we recommend using soil profile S_D for 2009 IBC seismic response evaluation.

Review of the Probabilistic Seismic Hazards Mapping Ground Motion Page[11] shows that the anticipated peak bedrock (soft rock - site class C) acceleration at the project site is approximately 0.46g and the peak ground (alluvium - site class D) acceleration is approximately 0.48g. The peak accelerations are based on a 10% probability of being exceeded in 50 years and are expressed as a fraction of the acceleration due to gravity (g). Site coordinates used for this assessment are longitude -121.995 and latitude 37.388.

8.3 Foundation Design

In general, the planned improvements will consist of constructing floodwalls on existing levees and creek banks and a reinforced concrete box culvert (RCB). We recommend the following allowable bearing pressures (presented in pounds per square feet, psf) for use in design of both floodwalls and RCB:

Loading Condition	Allowable Bearing Pressure (psf)
Dead Load	2,000
Dead plus Live Loads	3,000
Total Loads, including wind or seismic	4,000

For floodwalls, it is important that the footing is set back from the slope face of creek banks to protect the footing from the effects of erosion and/or shallow slumps that may occur over time. Floodwall footings should be offset from the face of adjacent slopes at a minimum of 1/3 of the slope height (H) where slopes are 45 degrees or less. For adjacent slopes greater than 45 degrees, the footing should be offset at a minimum 1/3 H from a 45 degree plane projected from the toe of the slope.

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[11] California Geological Survey, Department of Conservation. October, 2009. Probabilistic Seismic Hazards Assessment. Website: http://www.conservation.ca.gov/cgs/

8.3.1 Foundation Preparation

We estimated up to 2 inches of total settlement and about 1 inch of differential settlement due to the induced floodwall loadings. To reduce settlement, we recommend the base of the footing excavation be compacted using a smooth wheel roller with at least 6 passes. The footing excavation should also be clean and free of loose debris or materials and then lightly moisture conditioned just before pouring concrete to prevent the adjacent soil from drawing moisture from the concrete.

To create a smooth and uniform pad for the RCB slab, we recommend placing at least 12 inches of compacted Class II aggregate material below the base of the RCB. The Contractor should over excavate 12 inches of soil below the culvert and then backfill with Class II aggregate base. Aggregate base material should meet our recommendations for structural fill. Material requirements, placement, and compaction of structural fill are discussed in Section 8.1.5 and 8.1.6 of this report.

8.4 Lateral Earth Pressures

8.4.1 Retaining Structures

Buried or retaining structures should be designed using the following equivalent fluid pressures (presented in pounds per cubic feet, pcf):

Loading Condition	Allowable Lateral Load		REMARKS
	Above Water	Below Water (Saturated)	
Active Soil Pressure	50	25+63 (H2O) = 88	
At-Rest Soil Pressure	65	35+63 (H2O) = 98	
Passive Soil Pressure	300	150+63 (H2O) = 213	Maximum lateral value of 3000PSF

Passive pressure in the upper one foot of soil should be ignored if not confined by a slab-on-grade or pavement. Where floodwalls are located adjacent to a slope face, passive resistance should be ignored for soils between the slope face and footing. The recommended passive resistance includes a safety factor reduction of 1.5. Resistance to lateral loads may also be provided by friction along the base of foundations. A friction coefficient of 0.35 times the dead load may be used to evaluate ultimate frictional resistance along the bottom of foundations set on levee fill. A friction coefficient of 0.45 times the dead load may be used to evaluate ultimate frictional resistance along the bottom of foundations set on compacted aggregate base materials.

Wall backfill materials should be compacted to a minimum of 90 percent relative compaction in accordance with ASTM 1557 but no greater than 95 percent. Otherwise, the lateral earth pressures provided in the previous table may be much higher.

Where traffic and/or heavy equipment loads are anticipated, structures should be designed for an additional uniform lateral pressure equal to 100 psf over the entire height of the structure. For general surcharge loads behind the structure, a uniform lateral pressure of 0.5 times the anticipated surcharge load applied over the full height of the wall may be used. For seismic design, the buried or retaining structure should be designed to resist an additional uniform load equal to 20H psf, where H equals the height of soil retained by the structure in feet.

Although groundwater was observed about 39 feet bgs or deeper in our borings along the U-frame channel, we did encounter layers of clay that can create perched groundwater conditions behind the walls during the rainy season. Therefore, we recommend the U-frame channel walls should be fully drained to prevent the buildup of hydrostatic pressure. Wall drainage should consist of a either a drain rock layer at least 12 inches thick that extends to within 1 foot of the ground surface or a geosynthetic drainage panel that can capture the water and direct it to a perforated pipe drain or weep hole. A four-inch-diameter perforated PVC pipe (with perforations down) or weep holes should be installed at the base of the wall. The PVC pipe should sit on a two-inch-thick bed of drain rock. The pipe should be sloped to drain by gravity to a suitable drainage facility. To prevent surface infiltration behind the wall, the top 12 inches of backfill should be an impervious material such as compacted clay.

8.4.2 Reinforced Concrete Box Culvert

We understand the structural engineer will be following CalTrans standards to design the box culvert at the intersection of Hale Creek and Mountain View Avenue. Therefore, the following lateral earth densities should be used for design of reinforced concrete box (presented in pounds per cubic feet, pcf).

Loading Condition	For Vertical Earth Pressure (pcf)	For Lateral Earth Pressure (pcf)
Condition 1	140	100
Condition 2	140	35

The two loading conditions presented in the above table should be applied separately, and the resulting maximum moments should be applied for structural design. Condition 1 also satisfies the saturated fill situation. When the box culvert is at grade, or with a cover equal to or less than 2 feet, the wheel loads should be distributed into the culvert roof as concentrated loads. The box slab should be designed for wheel loads based on the HS 20-44 truck. Wheel live loads to the culvert invert is assumed as a uniform load applied transversely across the width and 7 feet longitudinally along the length of the box culvert. CalTrans no longer applies additional lateral loads from surcharge because the two loadings conditions are believed to be the more conservative design. Additionally, based on extensive research, CalTrans no longer considers seismic loads for underground structures.

8.4.3 Temporary Support

Structures used for temporary support during construction should be designed using apparent lateral earth pressures. We understand helical soil anchors will be used to support the sides of the U-frame channel while the concrete invert slab is deepened between C-Line Stations 251+25 and 261+85. The following apparent lateral earth pressure distribution, in terms of the excavation height (H) in feet, should be used for design of the anchors:

Depth below Excavation (feet)	Earth Pressure (psf)
0	0
¼ H	48 x H
¾ H	48 x H
H	0

Each soil anchor should support a maximum capacity of 30 kips (30,000 pounds) from the soil. According to the project structural engineer, the soil anchors are required for the short term only. Therefore, the maximum anchor capacity includes a safety factor of 1.5 whereas a safety factor of 2 is typical for permanent applications. The following length (L) of soil anchor, which consists of the blades only, is required to achieve 30 kips of soil capacity:

1. For a helical blade diameter of 8 inches, L = 10 feet (minimum)
2. For a helical blade diameter of 10 inches, L = 8 feet (minimum)

Each anchor rod shall extend a minimum of 7 feet beyond the active earth zone so that the anchor blades carrying the load do not interfere with the active earth zone. The active earth zone consists of the triangular area bounded by the vertical wall face, the horizontal backfill at the surface, and a line projected from the base of the wall extending 45 degrees up to the horizontal backfill behind the wall.

During installation of the helical anchors, the Contractor should verify pullout capacity (Q_u) using the following formula: $Q_u = K \times T$, where K is the torque to capacity ratio and T is the final installation torque.

8.5 Plan Review and Construction Observation

We recommend that we continue to participate in the project design phase and review the project plans and specifications to check that they are in conformance with the intent of our recommendations. During construction, we should observe site preparation, fill compaction, and grading to verify that earthwork is performed in accordance with our recommendations. Our services during construction will also permit us to detect unanticipated conditions and make recommendations for changes in design and construction, if appropriate.

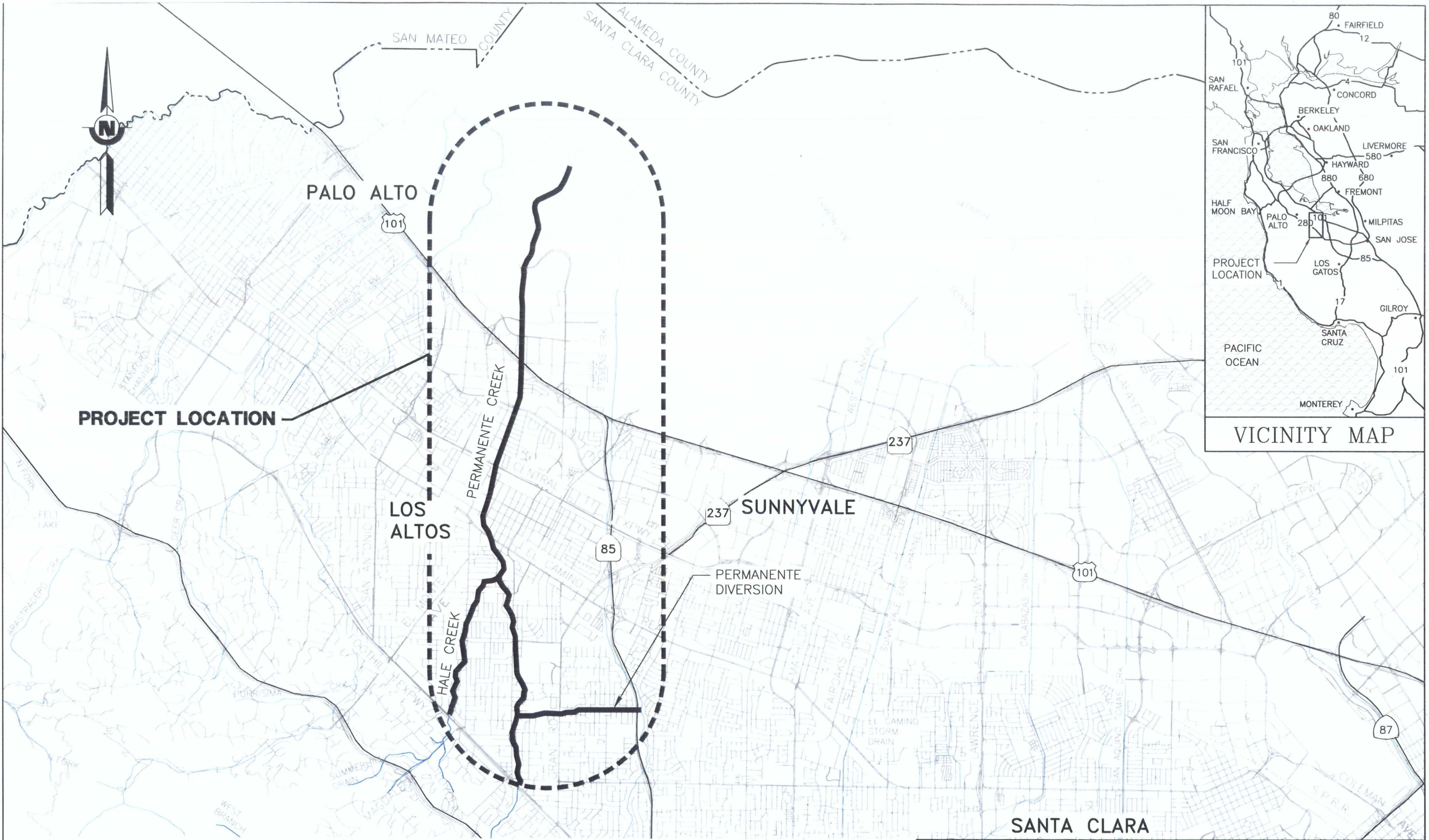
9.0 LIMITATIONS

Our services consist of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The analyses and recommendations contained in this report are based on the data obtained from fourteen exploratory borings and seven CPTs drilled by the District in years 2009 and 2003 and five exploratory borings drilled by Parikh in 2000, which indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on our analysis of the observed conditions. If conditions other than those described in this report are encountered, or changes in design occur we should be notified so that we can provide additional recommendations, if warranted.

Reliance on this report by others must be at their own risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for use by others of segregated portions of this report without our consultation.

PLATES



Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK
FLOOD PROTECTION PROJECT

LOCATION AND VICINITY MAP

SCALE	PROJECT NUMBER
1"=2500'	10244001
VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	PLATE 1

USERNAME: steikaro Wed 24 Mar 2010 03:58pm
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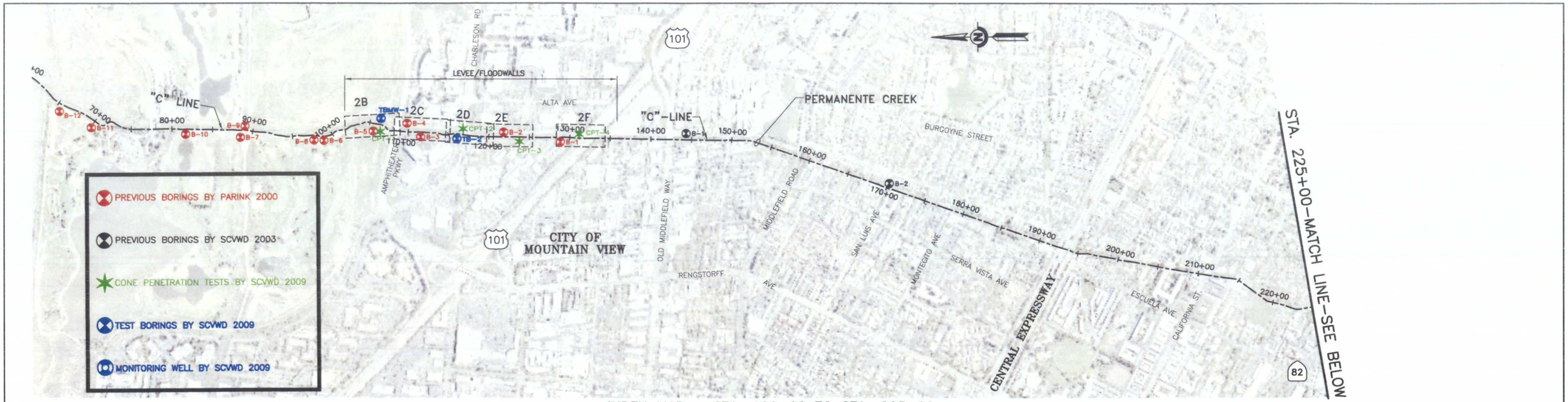
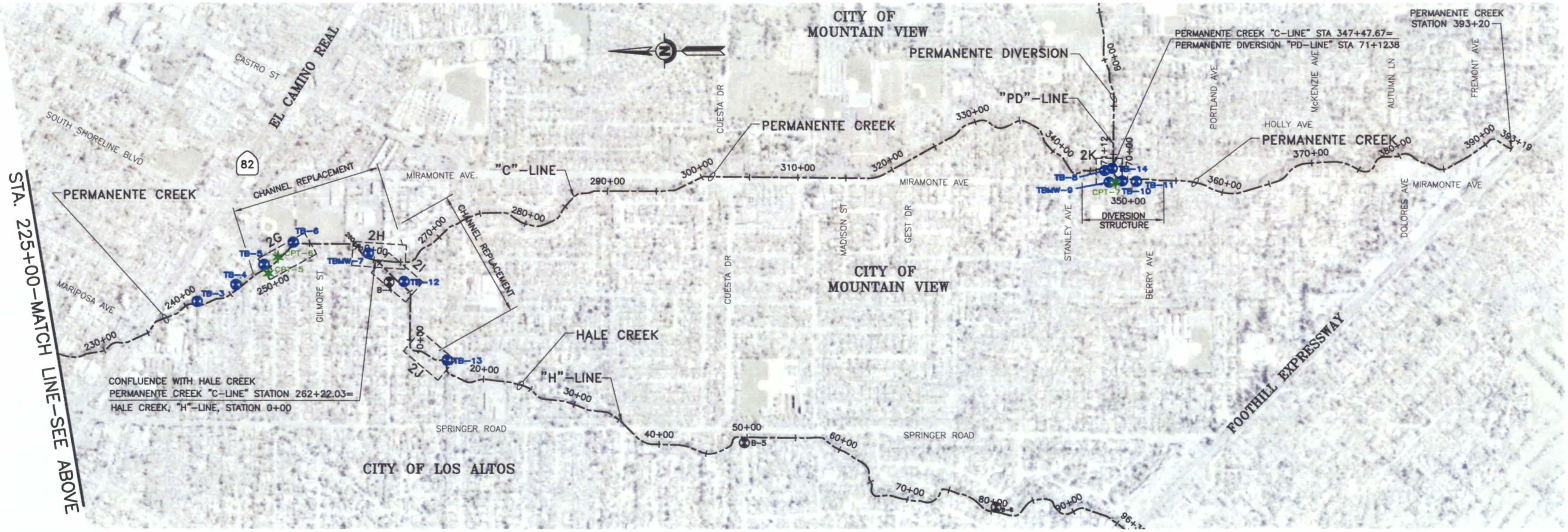


INDEX MAP – STA. 100+00 TO STA. 225+00

SCALE: 1" = 600'



INDEX MAP – STA. 225+00 TO STA. 393+20

SCALE: 1" = 600'

DOCUMENT NUMBER: PEM_LP-G-1023-XXXXX

REV	DESCRIPTION	DATE	APPR.	REFERENCE INFORMATION AND NOTES

DATE
MAR 2010
DESIGN
DRAWN
CHECKED
PROJECT ENGINEER
DATE

Santa Clara Valley Water District

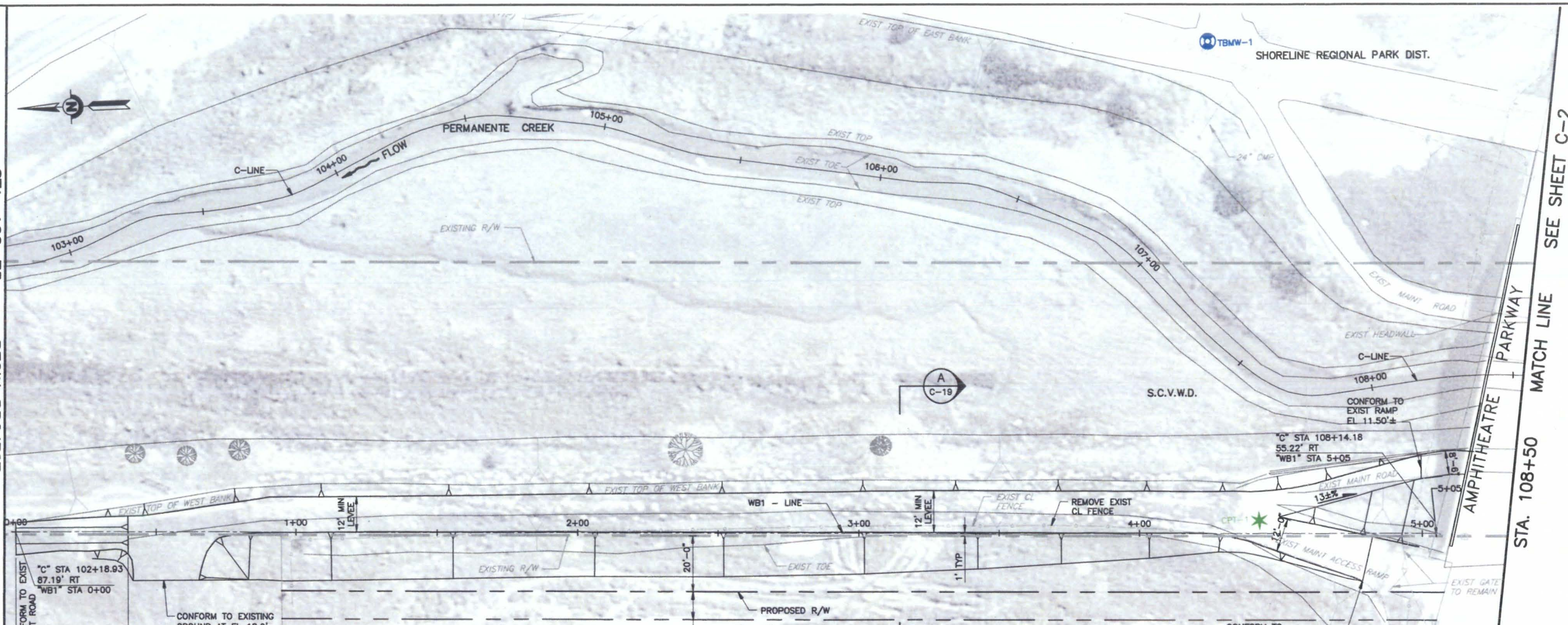
PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK FLOOD PROTECTION PROJECT

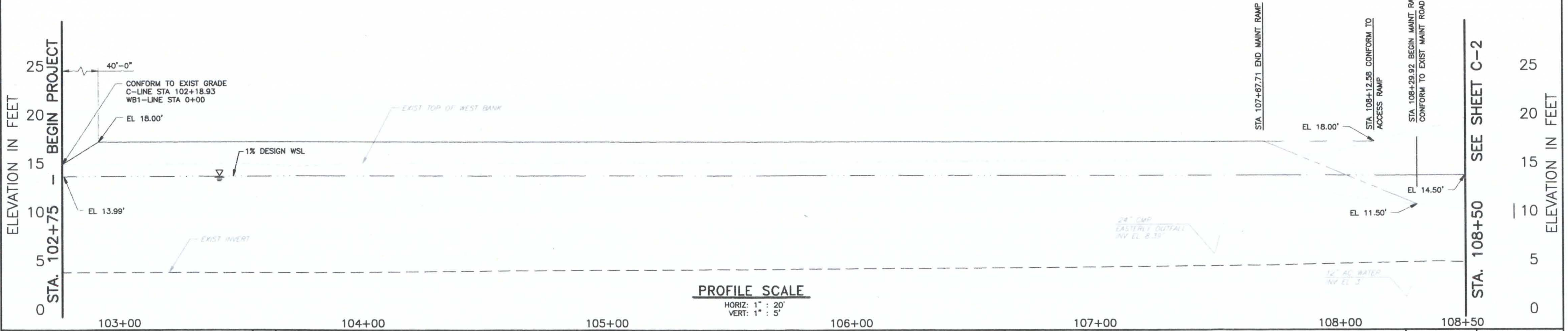
PERMANENTE CREEK, HALE CREEK AND PERMANENTE DIVERSION SOIL BORING INDEX MAP

SCALE
AS SHOWN
VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER
26244001
PLATE
2A



PLAN
SCALE: 1" = 20'


STA. 102+75 BEGIN PROJECT
ELEVATION IN FEET
25
20
15
10
5
0
40'-0"
CONFORM TO EXIST GRADE C-LINE STA 102+18.93 WB1-LINE STA 0+00
EL 18.00'
1% DESIGN WSL
EXIST TOP OF WEST BANK
EL 13.99'
EXIST INVERT
STA 107+67.71 END MAINT RAMP
STA 108+12.56 CONFORM TO ACCESS RAMP
STA 108+29.92 BEGIN MAINT RAMP CONFORM TO EXIST MAINT ROAD
EL 18.00'
EL 14.50'
EL 11.50'
24" CMP EASTERLY OUTFALL INV EL 8.39
12" AC WATER INV EL 3
SEE SHEET C-2 STA. 108+50
103+00
104+00
105+00
106+00
107+00
108+00
108+50

PROFILE SCALE
HORIZ: 1" : 20'
VERT: 1" : 5'

REV	DESCRIPTION	DATE	APPR.	REFERENCE INFORMATION AND NOTES
				1. DESIGN 1% FLOWRATE IS 1200 CFS

DATE
MAR 2010
DESIGN
DRAWN
CHECKED
PROJECT ENGINEER DATE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

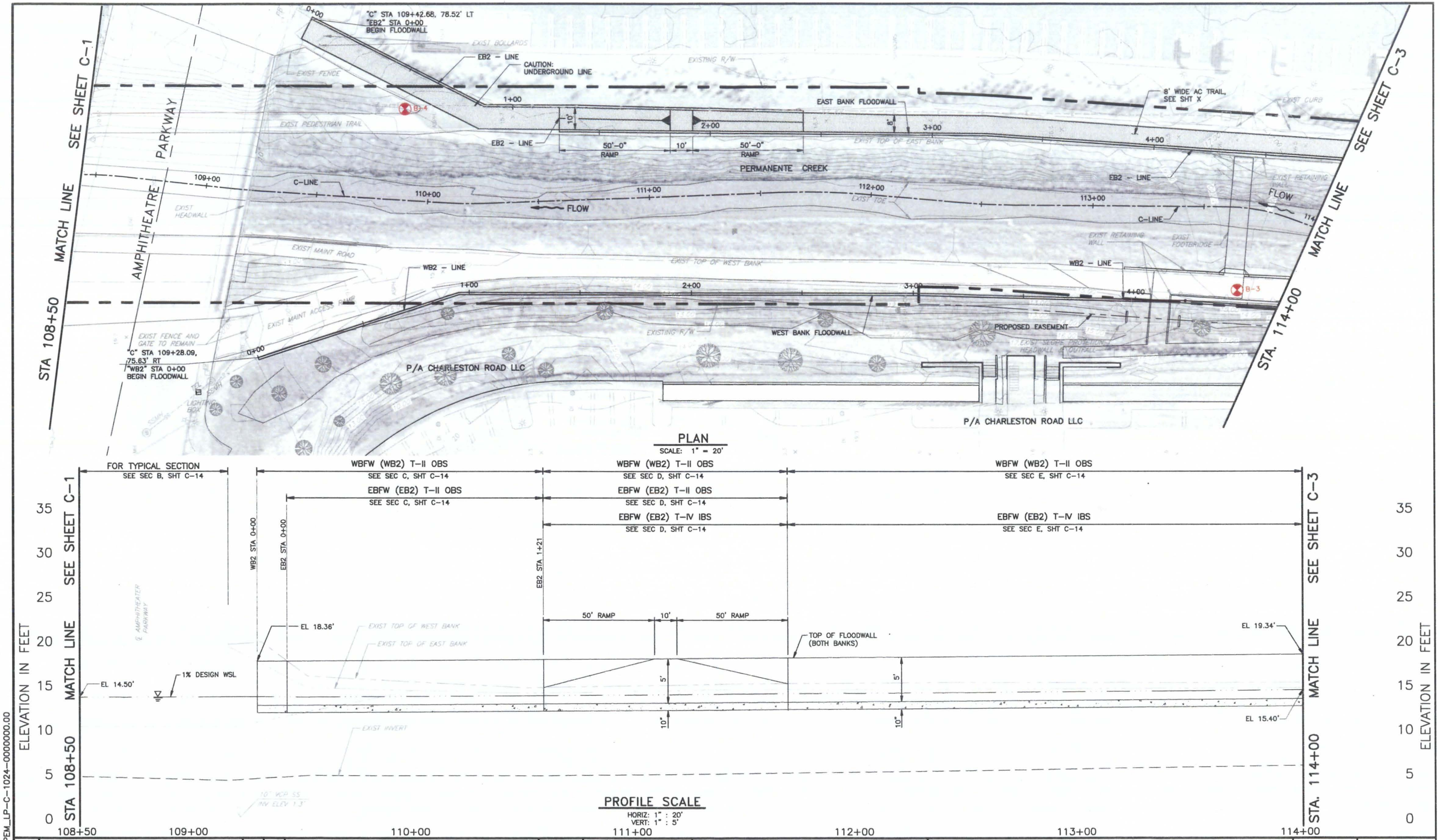
PERMANENTE CREEK
FLOOD PROTECTION PROJECT

PERMANENTE CREEK
SOIL BORING SITE PLAN

SCALE
AS SHOWN
VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER
26244001
PLATE
2B

DOCUMENT NUMBER: PEM_LP-C-1024-0000000.00



REV	DESCRIPTION	DATE	APPR.	REFERENCE INFORMATION AND NOTES
				1. DESIGN 1% FLOWRATE IS 1200 CFS

DATE: MAR 2010
DESIGN
DRAWN
CHECKED
PROJECT ENGINEER DATE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK
FLOOD PROTECTION PROJECT

PERMANENTE CREEK
SOIL BORING SITE PLAN

SCALE: AS SHOWN
VERIFY SCALES
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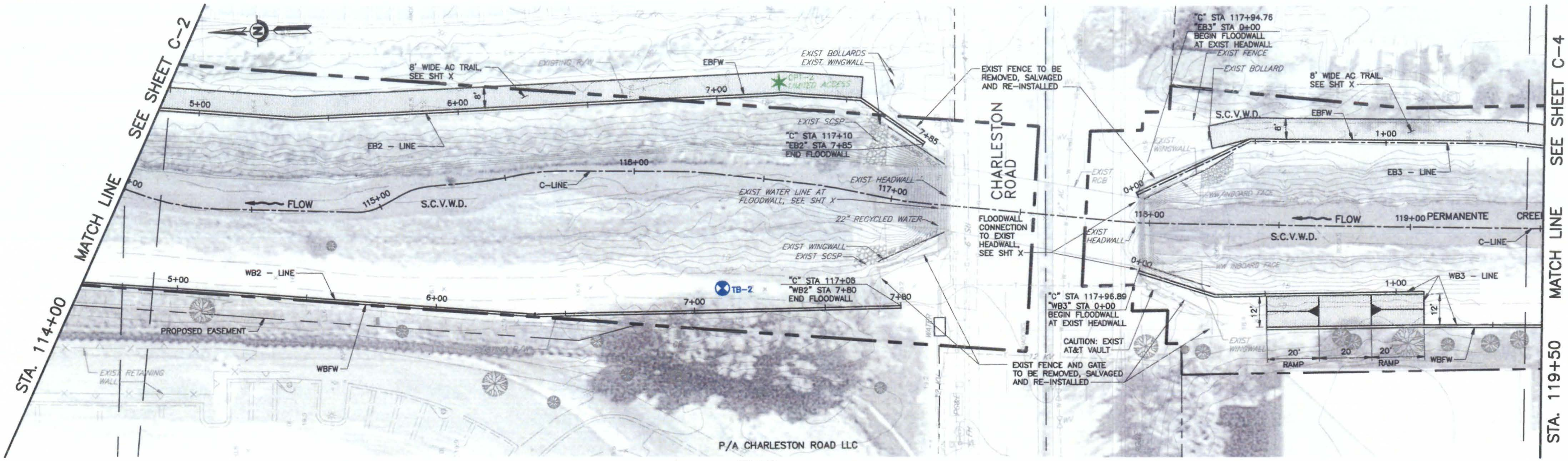
BAR IS ONE INCH ON ORIGINAL DRAWING
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER: 26244001
PLATE: 2C



PLAN

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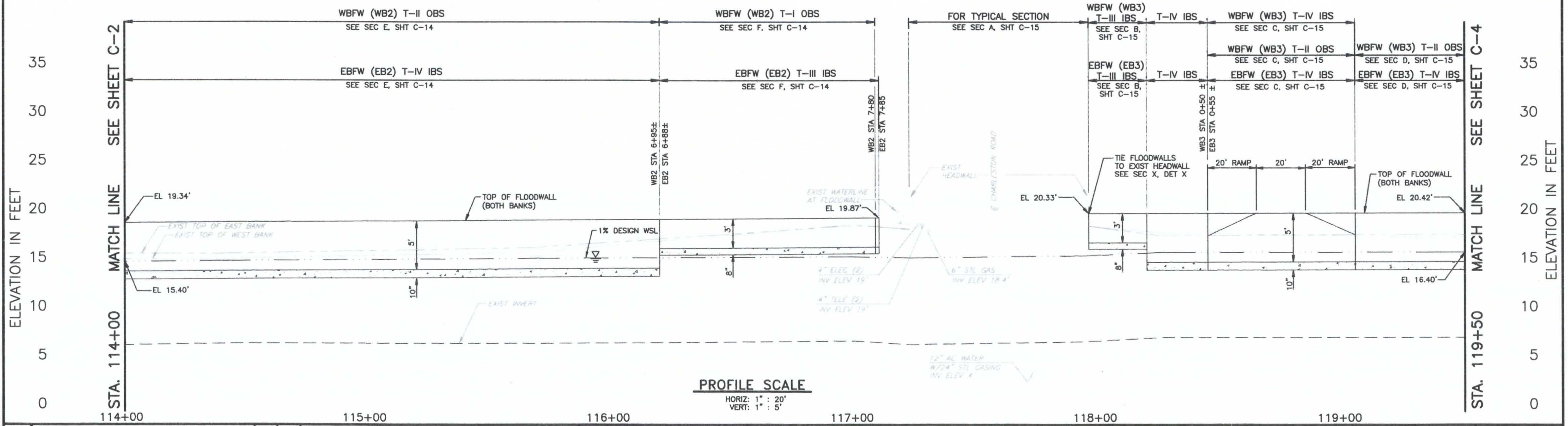


PROFILE SCALE

HORIZ: 1" : 20'
VERT: 1" : 5'

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES

DATE
MAR 2010
DESIGN
DRAWN
CHECKED
PROJECT ENGINEER
DATE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK FLOOD PROTECTION PROJECT

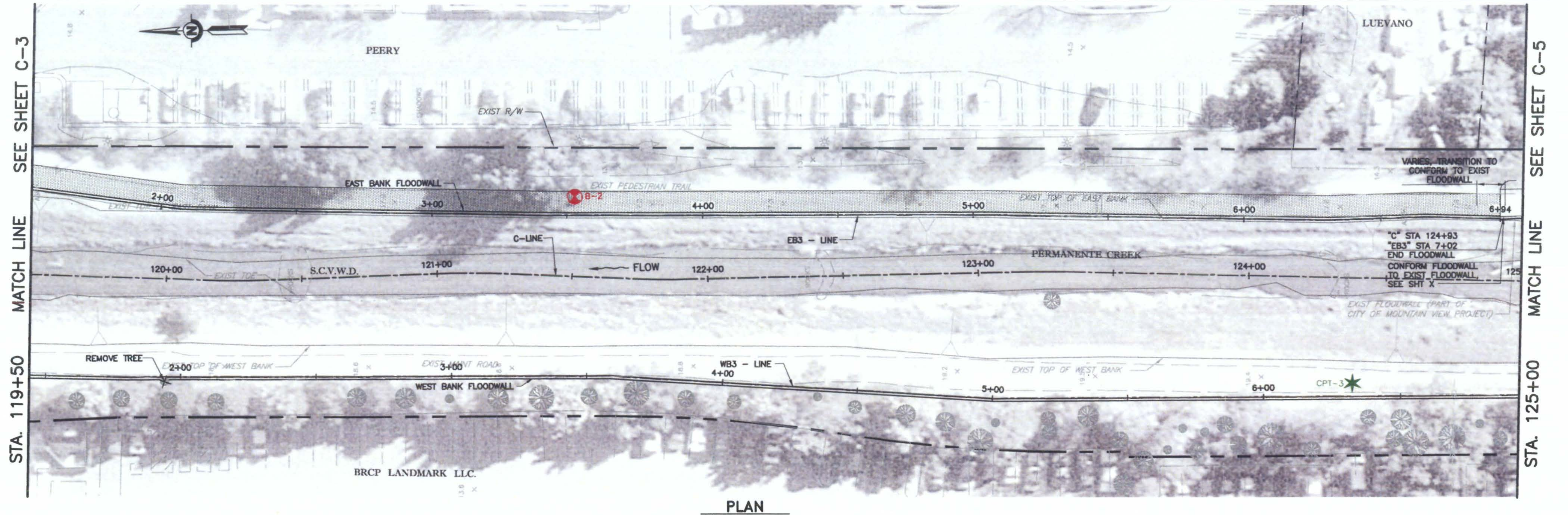
PERMANENTE CREEK SOIL BORING SITE PLAN

SCALE
AS SHOWN

VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER
26244001

PLATE
2D



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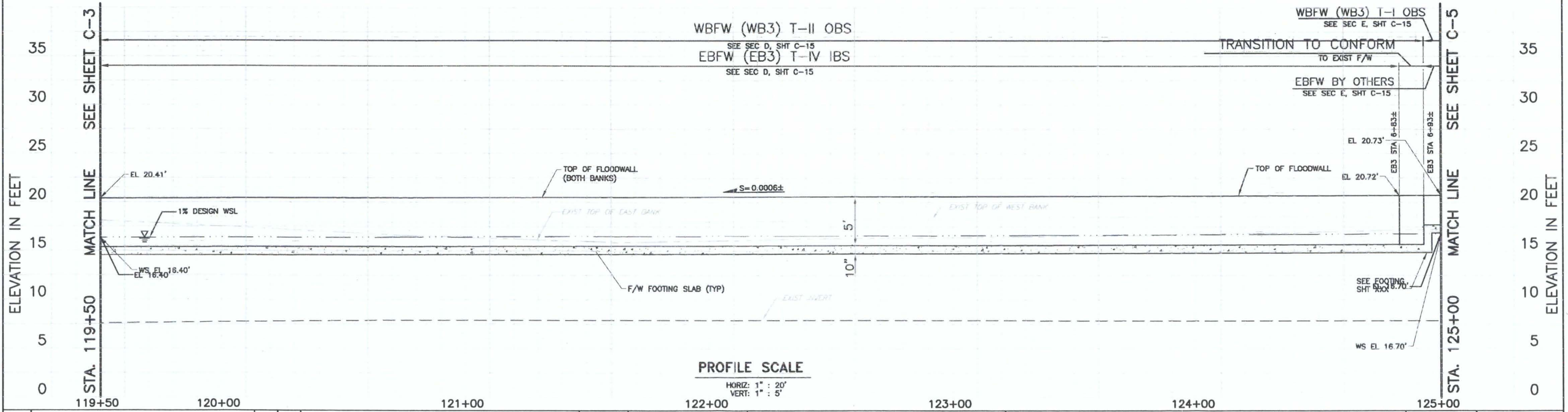

PROFILE SCALE

HORIZ: 1" : 20'
VERT: 1" : 5'

DOCUMENT NUMBER: PEM_LP-C-1024-XXXXX

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES
1. DESIGN 1% FLOWRATE IS 1200 CFS.

DATE
MAR 2010
DESIGN
DRAWN
CHECKED
PROJECT ENGINEER DATE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK
FLOOD PROTECTION PROJECT

PERMANENTE CREEK
SOIL BORING SITE PLAN

SCALE
AS SHOWN

VERIFY SCALES
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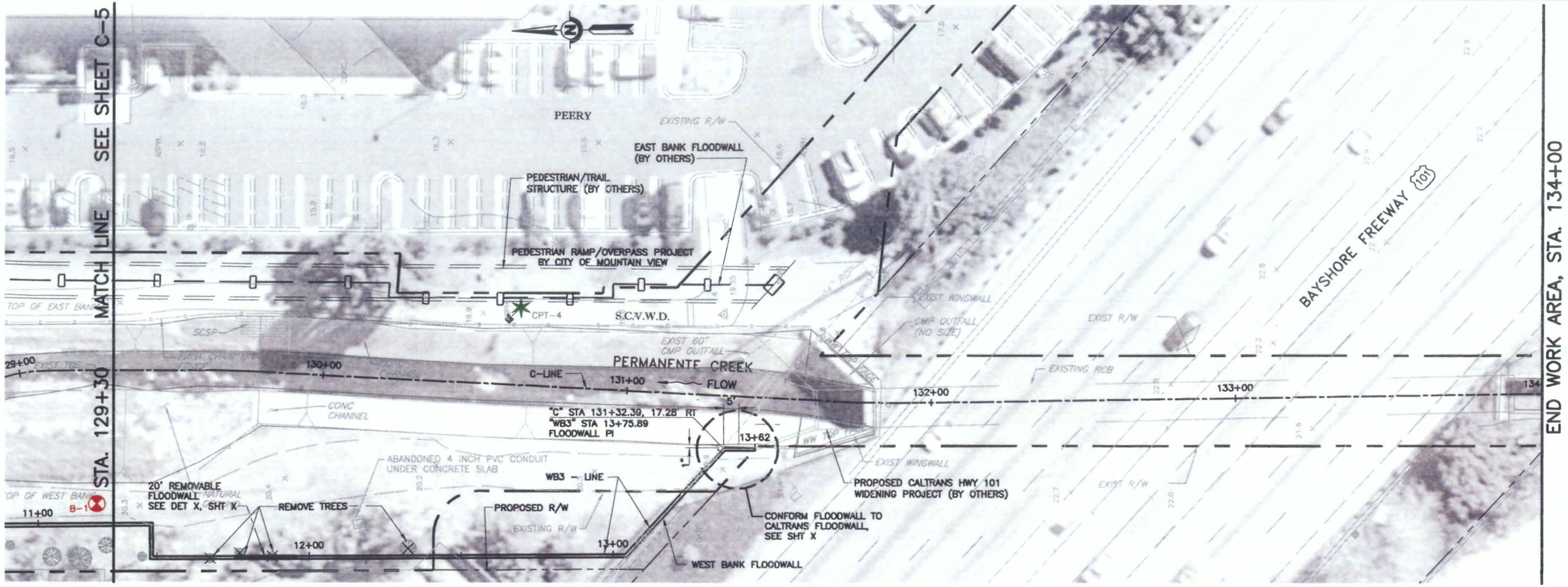
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER
26244001

PLATE
2E



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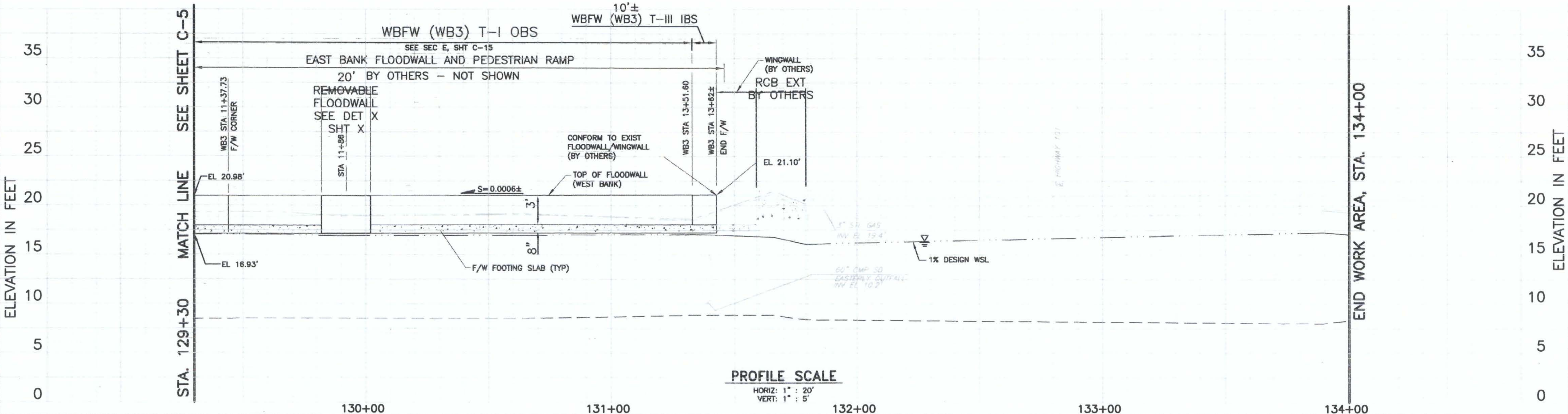

PROFILE SCALE
HORIZ: 1" : 20'
VERT: 1" : 5'

DOCUMENT NUMBER: PEM_LP-C-1023-XXXXXX

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES
1. DESIGN 1% FLOWRATE IS 1200 CFS
2.

DATE	MAR 2010
DESIGN	
DRAWN	
CHECKED	

PROJECT ENGINEER DATE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK FLOOD PROTECTION PROJECT

PERMANENTE CREEK SOIL BORING SITE PLAN

SCALE: AS SHOWN

VERIFY SCALES
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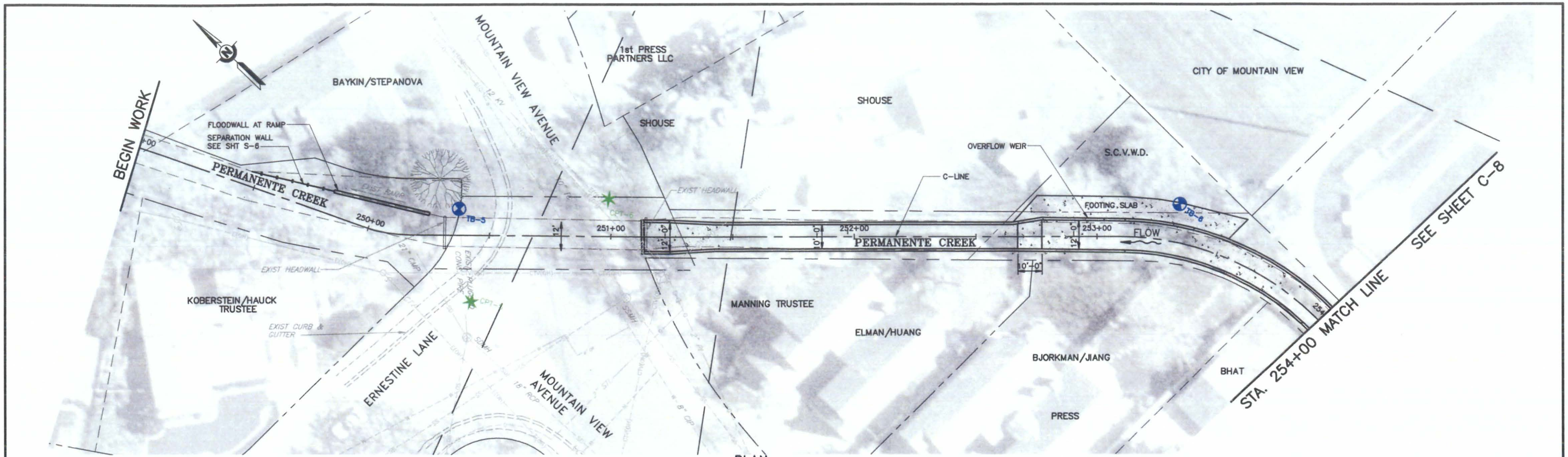
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER: 26244001

PLATE: 2F



PLAN

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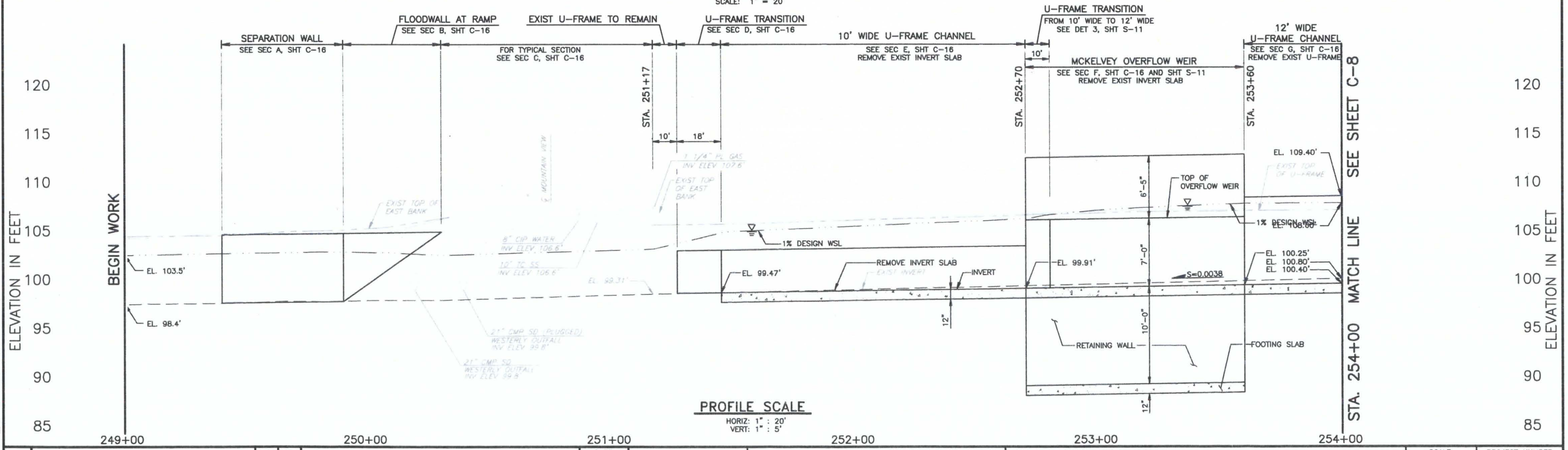

SEPARATION WALL SEE SEC A, SHT C-16
FLOODWALL AT RAMP SEE SEC B, SHT C-16
EXIST U-FRAME TO REMAIN
FOR TYPICAL SECTION SEE SEC C, SHT C-16
U-FRAME TRANSITION SEE SEC D, SHT C-16
10' WIDE U-FRAME CHANNEL SEE SEC E, SHT C-16 REMOVE EXIST INVERT SLAB
U-FRAME TRANSITION FROM 10' WIDE TO 12' WIDE SEE DET 3, SHT S-11
10'
MCKELVEY OVERFLOW WEIR SEE SEC F, SHT C-16 AND SHT S-11 REMOVE EXIST INVERT SLAB
12' WIDE U-FRAME CHANNEL SEE SEC G, SHT C-16 REMOVE EXIST U-FRAME
STA. 251+17
STA. 252+70
STA. 253+60
10'
18'
BEGIN WORK
EL. 103.5'
EL. 98.4'
EXIST TOP OF EAST BANK
8" CIP WATER INV ELEV 106.6'
10" PC SS INV ELEV 106.6'
EL. 99.31'
1 1/4" PL GAS INV ELEV 107.6
EXIST TOP OF EAST BANK
1% DESIGN WSL
EL. 99.47'
REMOVE INVERT SLAB
EXIST INVERT
INVERT
12"
EL. 99.91'
6'-5"
7'-0"
10'-0"
TOP OF OVERFLOW WEIR
S=0.0038
RETAINING WALL
FOOTING SLAB
12"
EL. 109.40'
EXIST TOP OF U-FRAME
1% DESIGN WSL
EL. 106.00'
EL. 100.25'
EL. 100.80'
EL. 100.40'
21" CMP SD (PLUGGED) WESTERLY OUTFALL INV ELEV 99.8'
21" CMP SD WESTERLY OUTFALL INV ELEV 99.9
ELEVATION IN FEET
120
115
110
105
100
95
90
85
249+00
250+00
251+00
252+00
253+00
254+00
STA. 254+00 MATCH LINE SEE SHEET C-8

PROFILE SCALE

HORIZ: 1" : 20'
VERT: 1" : 5'

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES

1. DESIGN 1% FLOWRATE IS 900 CFS

DATE
MAR 2010
DESIGN
DRAWN
CHECKED
PROJECT ENGINEER DATE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK FLOOD PROTECTION PROJECT

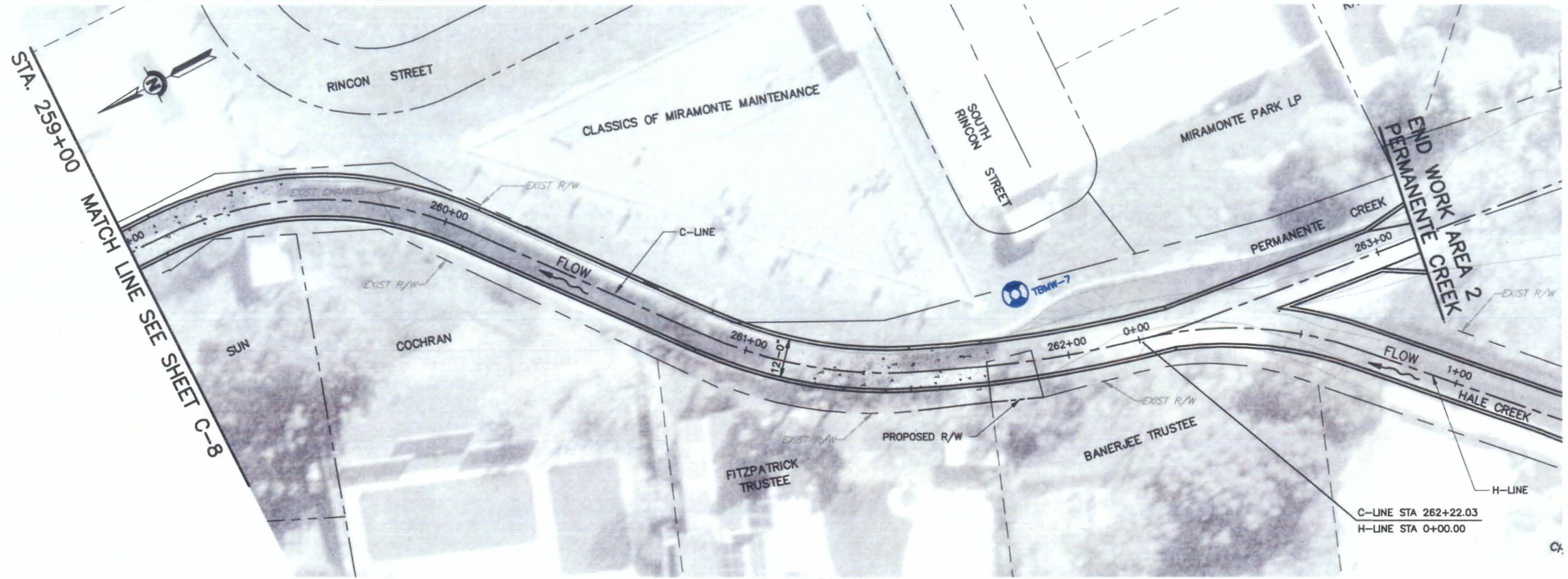
PERMANENTE CREEK SOIL BORING SITE PLAN

SCALE: AS SHOWN

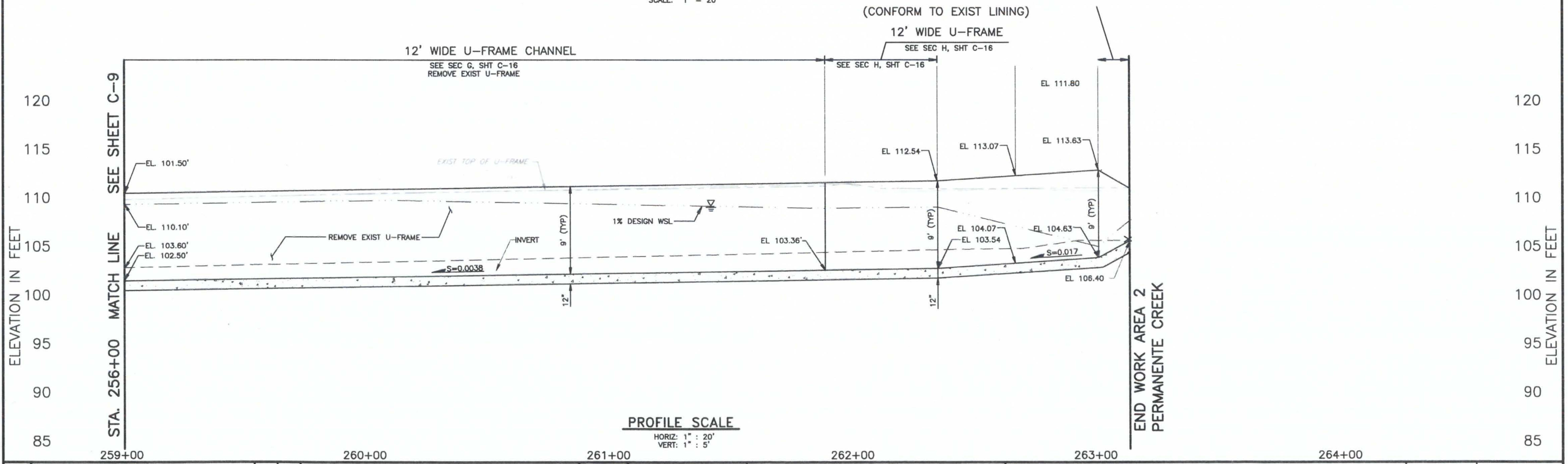
VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER: 26244001

PLATE: 2G



PLAN
SCALE: 1" = 20'



PROFILE SCALE
HORIZ: 1" : 20'
VERT: 1" : 5'

DOCUMENT NUMBER: PEM_LP-C-1024-0000000.00

REV	DESCRIPTION	DATE	APPR.	REFERENCE INFORMATION AND NOTES
				1. DESIGN 1% FLOWRATE IS 1300 CFS

DATE
MAR 2010
DESIGN
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Santa Clara Valley Water District

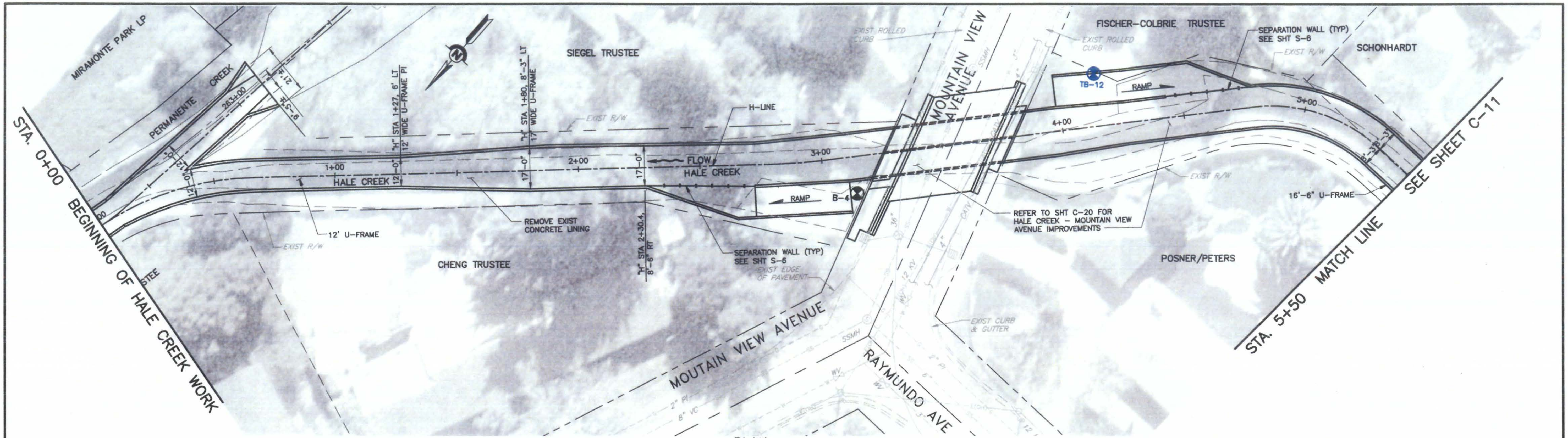
PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK FLOOD PROTECTION PROJECT

PERMANENTE CREEK SOIL BORING SITE PLAN

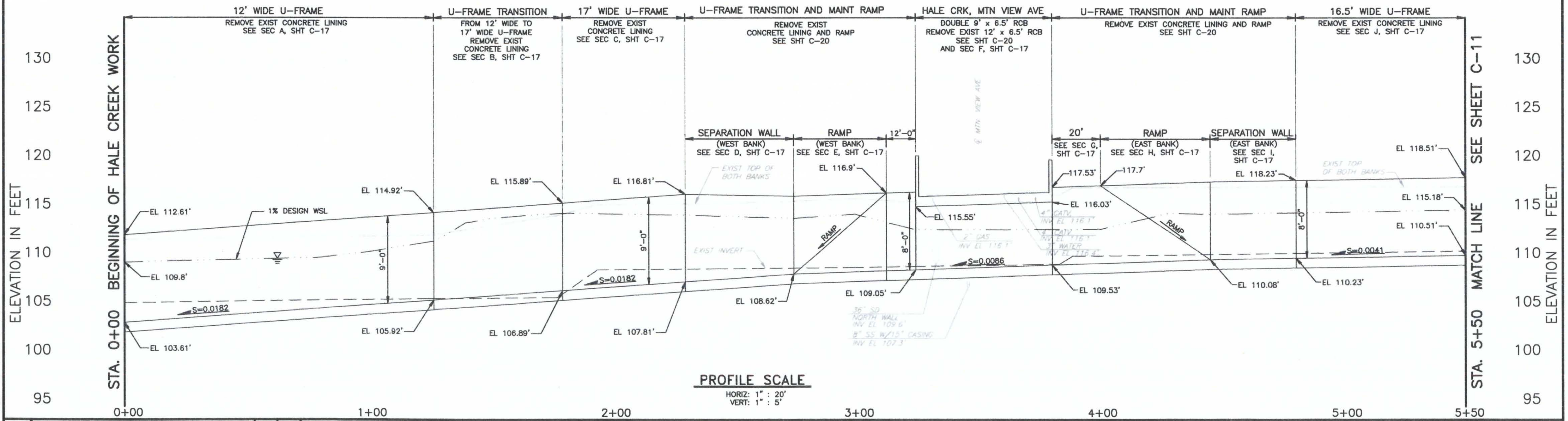
SCALE
AS SHOWN
VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER
26244001
PLATE
2H



PLAN

SCALE: 1" = 20'



PROFILE SCALE

HORIZ: 1" : 20'
VERT: 1" : 5'

REV	DESCRIPTION	DATE	APPR.	REFERENCE INFORMATION AND NOTES
				1. DESIGN 1% FLOWRATE IS 1100 CFS

DATE MAR 2010
DESIGN
DRAWN
CHECKED
PROJECT ENGINEER DATE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK FLOOD PROTECTION PROJECT

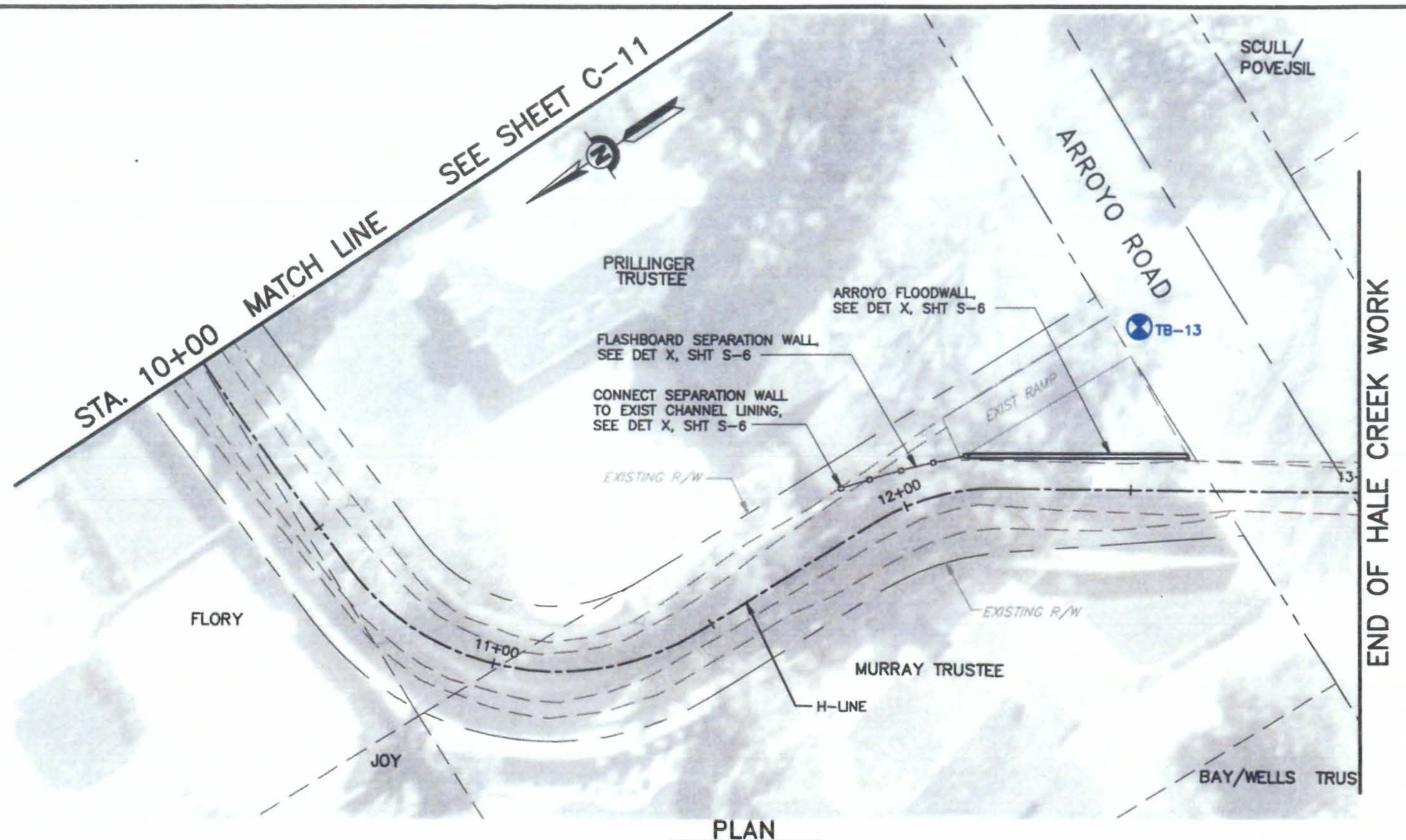
HALE CREEK SOIL BORING SITE PLAN

SCALE AS SHOWN
VERIFY SCALES 0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER 26244001

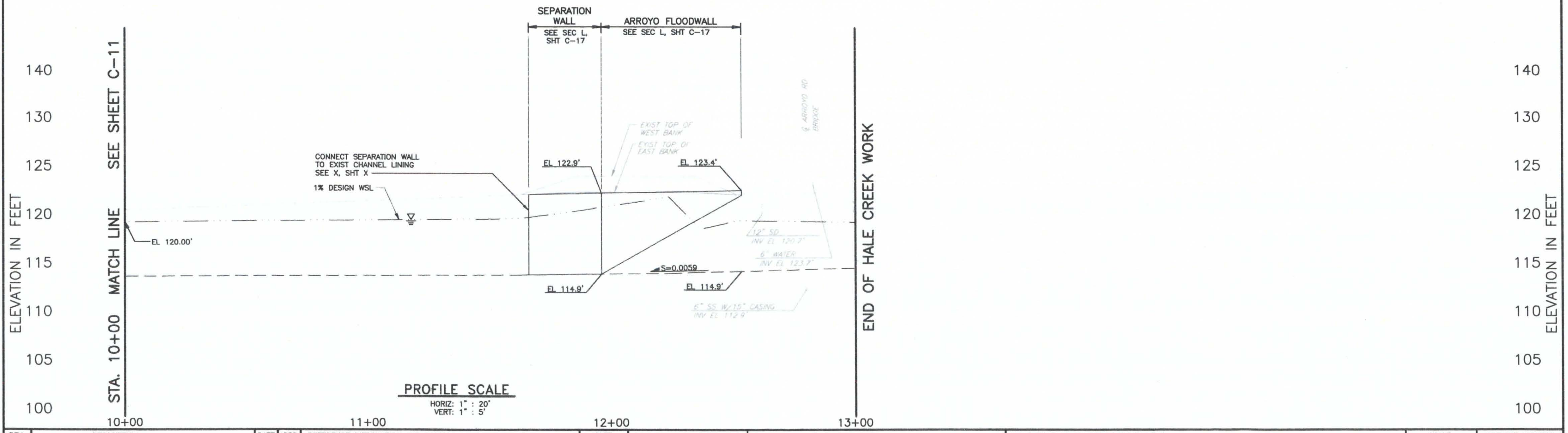
PLATE 2I

DOCUMENT NUMBER: PEM_LP-C-1024-XXXXXXX


STA. 10+00 MATCH LINE SEE SHEET C-11
N
SCULL/ POVEJSIL
ARROYO ROAD
PRILLINGER TRUSTEE
ARROYO FLOODWALL, SEE DET X, SHT S-6
TB-13
FLASHBOARD SEPARATION WALL, SEE DET X, SHT S-6
CONNECT SEPARATION WALL TO EXIST CHANNEL LINING, SEE DET X, SHT S-6
EXIST RAMP
EXISTING R/W
12+00
13
11+00
EXISTING R/W
FLORY
MURRAY TRUSTEE
H-LINE
JOY
BAY/WELLS TRUS
END OF HALE CREEK WORK

PLAN

SCALE: 1" = 20'



PROFILE SCALE

HORIZ: 1" : 20'
VERT: 1" : 5'

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES

1. DESIGN 1% FLOWRATE IS 1100 CFS

DATE: MAR 2010
DESIGN
DRAWN
CHECKED
PROJECT ENGINEER DATE

Santa Clara Valley Water District

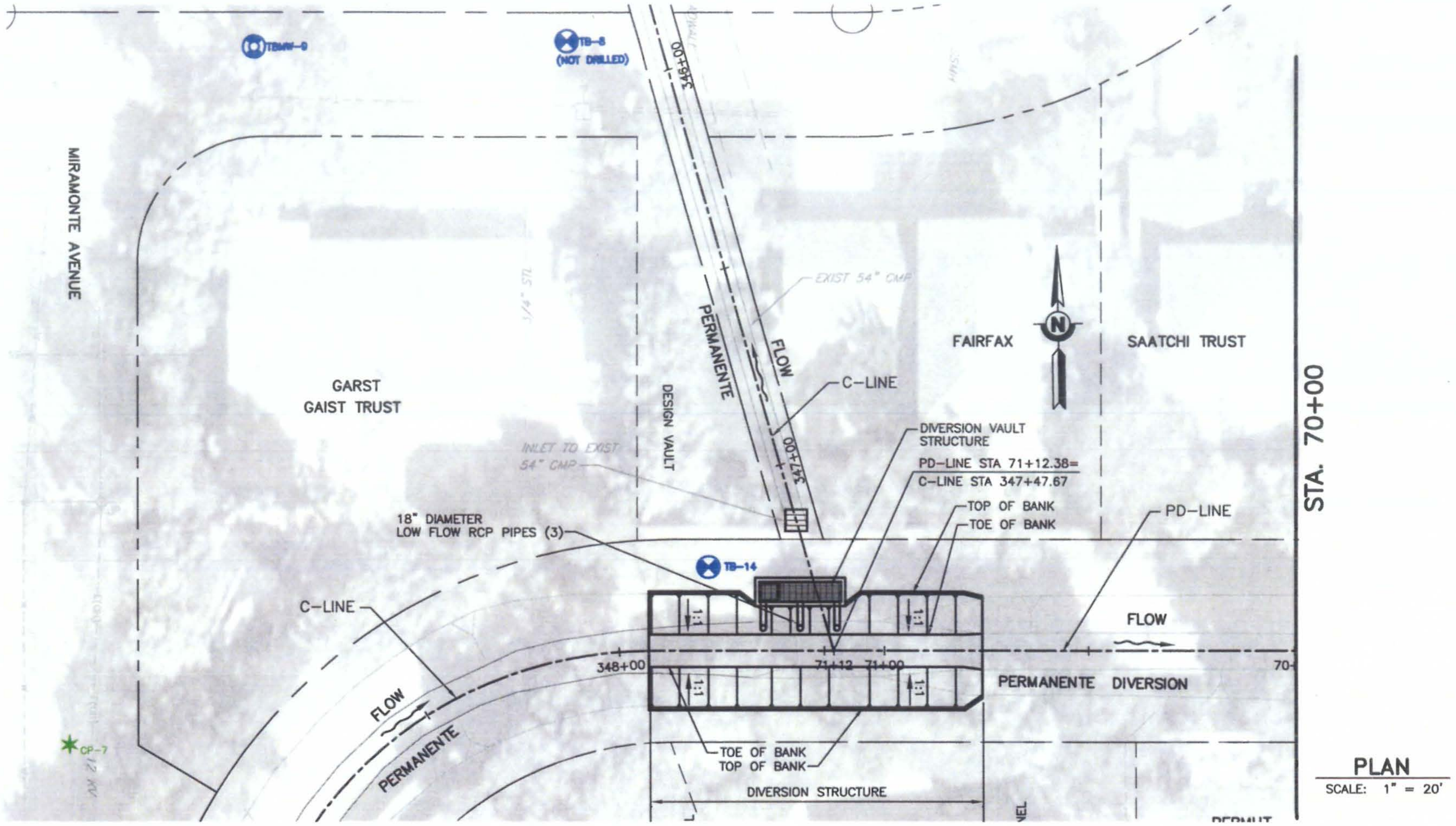
PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK
FLOOD PROTECTION PROJECT

HALE CREEK
SOIL BORING SITE PLAN

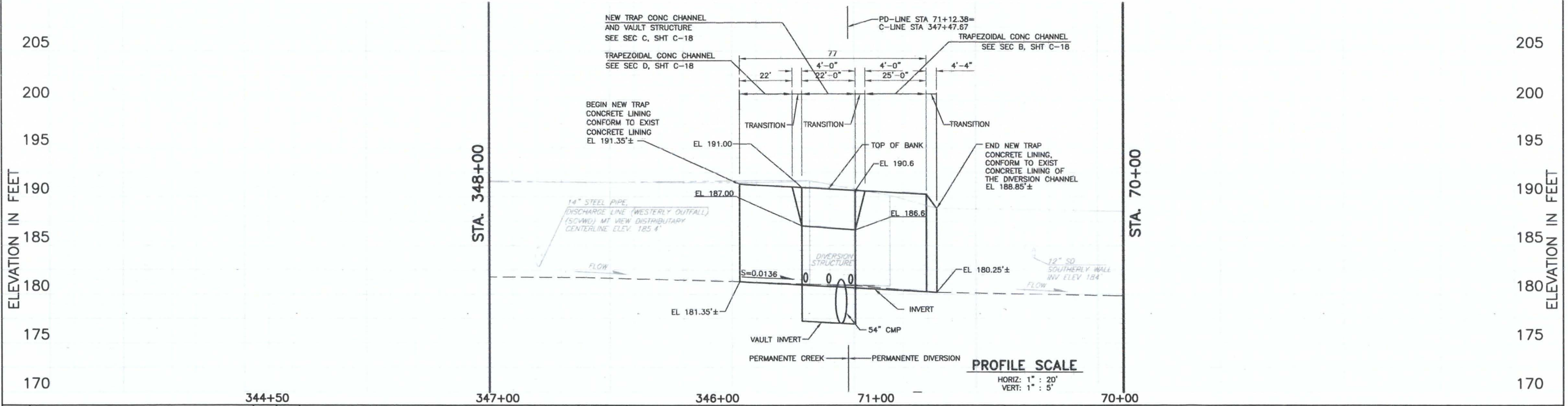
SCALE: AS SHOWN
VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER: 26244001
PLATE: 2J



PLAN

SCALE: 1" = 20'


205
200
195
190
185
180
175
170
ELEVATION IN FEET
STA. 348+00
STA. 70+00
NEW TRAP CONC CHANNEL AND VAULT STRUCTURE SEE SEC C, SHT C-18
TRAPEZOIDAL CONC CHANNEL SEE SEC D, SHT C-18
PD-LINE STA 71+12.38=
C-LINE STA 347+47.67
TRAPEZOIDAL CONC CHANNEL SEE SEC B, SHT C-18
77
22'
4'-0"
22'-0"
4'-0"
25'-0"
4'-4"
TRANSITION
TRANSITION
TRANSITION
BEGIN NEW TRAP CONCRETE LINING CONFORM TO EXIST CONCRETE LINING EL 191.35'±
EL 191.00
TOP OF BANK
EL 190.6
END NEW TRAP CONCRETE LINING, CONFORM TO EXIST CONCRETE LINING OF THE DIVERSION CHANNEL EL 188.85'±
EL 187.00
EL 186.6
14" STEEL PIPE, DISCHARGE LINE (WESTERLY OUTFALL) (SCVWD) MT VIEW DISTRIBUTARY CENTERLINE ELEV. 185.4'
DIVERSION STRUCTURE
FLOW
S=0.0136
EL 180.25'±
12" SD SOUTHERLY WALL INV ELEV 184
FLOW
EL 181.35'±
INVERT
54" CMP
VAULT INVERT
PERMANENTE CREEK
PERMANENTE DIVERSION
344+50
347+00
346+00
71+00
70+00

PROFILE SCALE

HORIZ: 1" : 20'
VERT: 1" : 5'

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES

1. PERMANENTE CREEK D/S OF STATION 347+50: DESIGN 1% FLOWRATE IS 50 CFS
2. PERMANENTE CREEK U/S OF STATION 347+50: DESIGN 1% FLOWRATE IS 2000 CFS
3. PERMANENTE DIVERSION D/S OF STATION 71+12: DESIGN 1% FLOWRATE IS 1950 CFS

DATE
MAR 2010
DESIGN
DRAWN
CHECKED

PROJECT ENGINEER DATE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

PERMANENTE CREEK FLOOD PROTECTION PROJECT

PERMANENTE CREEK AND DIVERSION SOIL BORING SITE PLAN

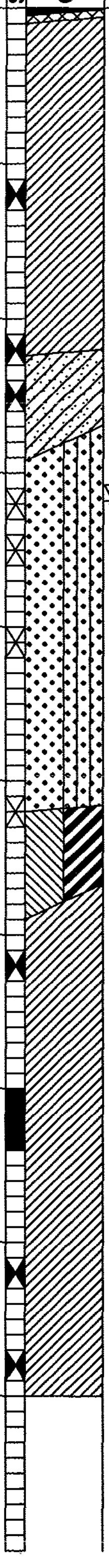
SCALE	PROJECT NUMBER
AS SHOWN	26244001
VERIFY SCALES	PLATE
0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	2K

DOCUMENT NUMBER: PEM_LP-C-1024-0000000.00

Boring Location DS of Amphitheatre Pkwy
Drill Rig Mobile B-53, 8" Hollow Stem
Top Elevation 15.1 Ft. **Date Drilled** 6/23/2009

Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)
						5
LL= 43%; PI= 25%	14*	2 3.5		18.9	105	
						10
	13*	2.5		17.9	103	
-#200 = 19%	17					
						15
	27*					
-#200 = 7%	29					
						20
MA: -#4 = 74% -#200 = 3%	9					
						25
	16					
						30
LL= 50%; PI= 28%	21*	3.5		25.2	100	
						35
Consol		2.5		26.7	96	
						40
	25*	3.25		26.6	98	
	26*	3.75		22.6	105	
						45

***Blows/foot converted to approximate standard penetration test blow counts.**



Soil Description

3" Asphalt Concrete
3" Aggregate Base
Dark Brown Lean CLAY (CL) with sand (Fill)
Medium stiff to very stiff, moist, fine to medium sand

Grayish Brown Clayey SAND with gravel (SC)
Medium dense, moist, medium to coarse sand, fine gravel

Gray Well Graded SAND with silt and gravel (SW-SM)
∇ Loose to medium dense, moist to wet
(6/23/2009 : 11:35)

Gray Lean to Fat CLAY (CL/CH)
Very stiff, moist, with organic material

Brown Lean CLAY (CL)
Very stiff, moist to wet, trace sand

Bottom of Boring at 45 Feet

Santa Clara Valley Water District

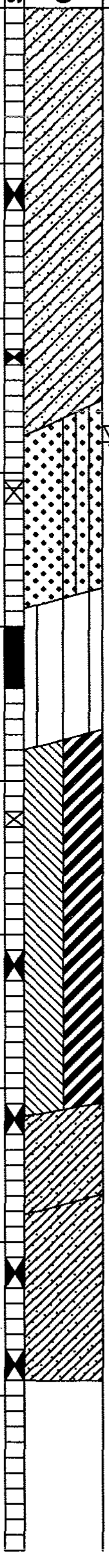
LOG OF BORING NO. TB-1

Permanente Creek Project
Mountain View and Los Altos, CA

PLATE 3

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location DS of Charleston Rd
Drill Rig Mobile B-53, 7" Hollow Stem
Top Elevation 16.9 Ft. **Date Drilled** 6/24/2009



Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							Dark Brown Clayey SAND (SC) - Fill Medium dense, dry to moist, medium to coarse sand
TxUU$_{(FM)}$= 2930 (600); -#200 = 44%	15*			13.0	121	5	
	6 *					10	
							(6/24/2009 : 09:30) Brown Well Graded SAND with silt and gravel (SW-SM) Very dense, moist to wet, fine to medium sand
MA: -#4 = 85% -#200 = 6%	39					15	
MA: -#4 = 100% -#200 = 73%; Consol		1		25.2	98	20	Brown SILT with sand (ML) Medium stiff to stiff, wet, fine sand
							Bluish Gray Lean to Fat CLAY with sand (CL-CH) Stiff to very stiff, moist to wet, fine to medium sand, bay mud
	12	1.25				25	
TxUU$_{(FM)}$= 2290 (2800)	19*	1.25		20.9	107	30	
LL= 28%; PI= 10%	12*	1.25		23.0	104	35	Brown Sandy Lean CLAY (CL) Stiff, wet, fine to medium sand
							Bluish Gray Lean CLAY with sand (CL) Stiff, wet, medium sand
	12*	1.75		25.6	100	40	
	24*	3.25		23.8	103		
						45	Bottom of Boring at 44.5 Feet

***Blows/foot converted to approximate standard penetration test blow counts.**

Santa Clara Valley Water District

LOG OF BORING NO. TB-2

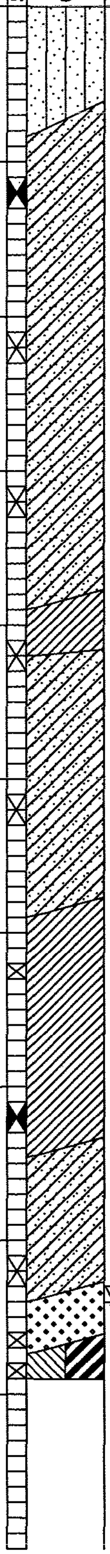
Permanente Creek Project
Mountain View and Los Altos, CA

PLATE
4

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location End of Park Dr
Drill Rig Marl M-57 Rhino, 6.25" Hollow Stem
Top Elevation 102.3 Ft. **Date Drilled** 7/2/2009

Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							Brown Sandy SILT (ML) Stiff, dry, medium sand
						5	Brown Clayey SAND with gravel (SC) Medium dense, moist, gravels to 1 inch, medium to coarse sand
MA: -#4 = 66% -#200 = 14%	15*			6.2	105		
						10	
	11						
						15	
MA: -#4 = 69% -#200 = 15%	28						
						20	Brown Sandy Lean CLAY (CL) Very stiff, moist, medium to coarse sand
	46						Brown Clayey SAND with gravel (SC) Medium dense to very dense, moist, gravels to 3/4 inch, medium to coarse sand
						25	
-#200 = 24%	16						
						30	Brown Lean CLAY (CL) Very stiff, moist, low to medium plasticity
	18	4					
						35	Increase in sand content at 35 feet
UC= 2310	16*	3.25		20.4	109		
							Brown Clayey SAND (SC) Medium dense, moist, medium to coarse sand, trace gravel
						40	
-#200 = 34%	15						∇ (7/2/2009 : 09:50) Brown Well Graded SAND (SW) Loose to medium dense, wet, medium to coarse sand
	20						Brown Sandy Lean to Fat CLAY (CL-CH) Very stiff, moist to wet, fine to medium sand, medium plasticity Bottom of Boring at 44.5 Feet
						45	



***Blows/foot converted to approximate standard penetration test blow counts.**

Santa Clara Valley Water District

LOG OF BORING NO. TB-3

Permanente Creek Project
Mountain View and Los Altos, CA

PLATE 5

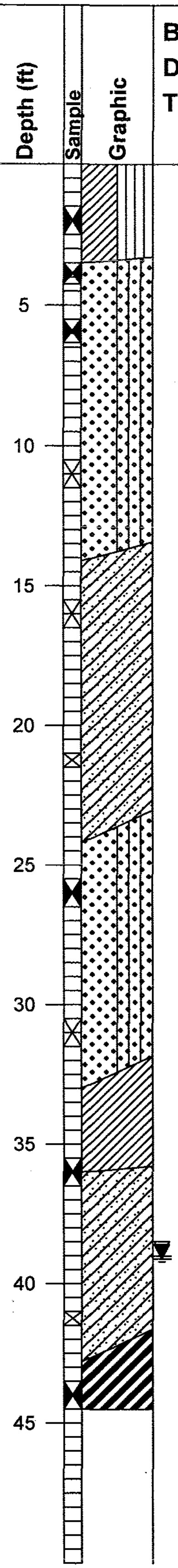
Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Rodert Property

Drill Rig Marl M-57 Rhino, 6.25" Hollow Stem

Top Elevation 103.2 Ft. **Date Drilled** 6/30/2009

Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Soil Description
						Brown Silty CLAY (CL-ML) - Fill Stiff to very stiff, dry
TxUU$_{(FM)}$= 1280 (200)	12*	2.		10.6	90	
	8*					Brown Well Graded SAND with silt and gravel (SW-SM) - Fill Loose to medium dense, dry, medium to coarse sand, gravels to 1/2 inch
MA: -#4 = 73% -#200 = 9%	14*					
	19					Gravels to 1.5 inches
						Brown Clayey SAND (SC) Medium dense to dense, moist, trace gravels to 3/4 inch
-#200 = 37%	30					Less gravels at 16 feet
-#200 = 24%	40					
MA: -#4 = 77% -#200 = 8%	67*					Brown Well Graded SAND with silt and gravel (SW-SM) Very dense, moist, medium to coarse sand, gravel to 3/4 inch
	59					
						Brown Sandy Lean CLAY (CL) Medium stiff, moist, medium to coarse sand
	8*			23.0	104	Brown Clayey SAND (SC) Loose, moist to wet, medium to coarse sand (6/30/2009 : 11:08) (6/30/2009 : 12:10)
	17					
	10*	1.5		26.5	98	Bluish Gray Fat CLAY (CH) Stiff, wet Bottom of Boring at 44.5 Feet



***Blows/foot converted to approximate standard penetration test blow counts.**

Santa Clara Valley Water District

LOG OF BORING NO. TB-4

Permanente Creek Project
Mountain View and Los Altos, CA

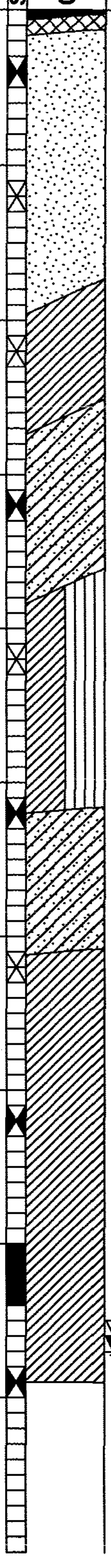
PLATE
6

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Corner of Mountain View Ave and Ernestine Ln

Drill Rig Marl M-57 Rhino, 6.25" Hollow Stem

Top Elevation 107.6 Ft. **Date Drilled** 6/29/2009



Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							4 inches Asphalt Concrete 6 inches Aggregate Base Brown Poorly Graded SAND (SP) - Fill Loose, moist, medium sand
MA: -#4 = 100% -#200 = 5%	12*						
						5	
	6						
						10	Brown Lean CLAY with sand (CL) Hard, dry to moist, fine to medium sand
	20	>4.5					
							Brown Clayey SAND (SC) Medium dense, moist, trace fine to medium gravel
						15	
$TxUU_{(FM)}$= 3590 (1600); -#200 = 40%	20*			13.1 12.2	113 118		
						20	Light Brown Silty CLAY with sand (CL-ML) Very stiff, moist, fine to medium sand
	23	4					
						25	
	27*			16.9	109		Brown Clayey SAND with gravel (SC) Medium dense, moist
						30	
	22	4					Light Brown Lean CLAY (CL) Stiff to very stiff, dry to moist
						35	
UC= 1250	18*	3.25		24.6	100		
						40	with medium to coarse sand at 40 feet
Consol		3		19.4	110		
							(6/29/2009 : 14:00)
	24*	2.25		28.0	96	45	(6/29/2009 : 14:10) Bottom of Boring at 44.5 Feet

***Blows/foot converted to approximate standard penetration test blow counts.**

Santa Clara Valley Water District

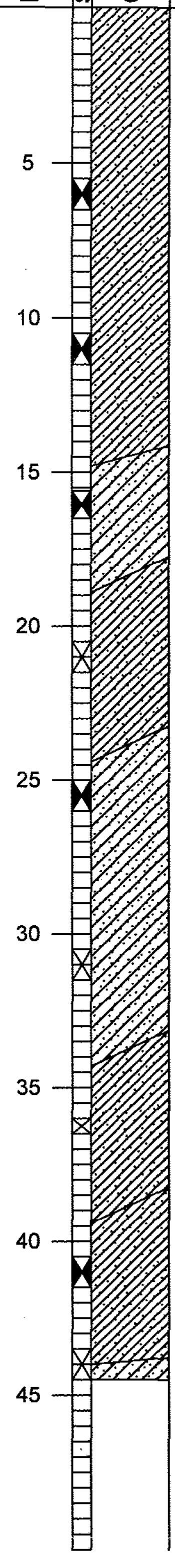
LOG OF BORING NO. TB-5

Permanente Creek Project
Mountain View and Los Altos, CA

PLATE 7

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Next to McKelvey Park
Drill Rig Marl M-57 Rhino, 6.25" Hollow Stem
Top Elevation 108.9 Ft. **Date Drilled** 7/2/2009



Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							Brown Sandy Lean CLAY (CL) Stiff to hard, dry to moist, fine sand
$TxUU_{(FM)}$= 1320 (600)	11*	2.25		10.9	98	5	
-#200 = 60%	25*	>4.5		10.4	106	10	
-#200 = 28%	14*					15	Brown Clayey SAND (SC) Loose, dry to moist, fine to medium sand, trace gravel
	26	4.5				20	Brown Sandy Lean CLAY (CL) Hard, dry to moist, fine to medium sand
	13*			28.4	92	25	Brown Clayey SAND (SC) Medium dense, moist, fine to medium sand
	27					30	Sandy Lean CLAY lense from 30.5 to 30.8 feet
	23	3.5				35	Brown Lean CLAY with sand (CL) Very stiff, moist, fine to medium sand
LL= 39%; PI= 19%	18*	2.5		24.1	102	40	Brown Sandy Lean CLAY (CL) Very stiff, moist, fine to medium sand
	39					45	Brown Clayey SAND (SC) Dense, moist, medium to coarse sand Bottom of Boring at 44.5 Feet

***Blows/foot converted to approximate standard penetration test blow counts.**

Santa Clara Valley Water District

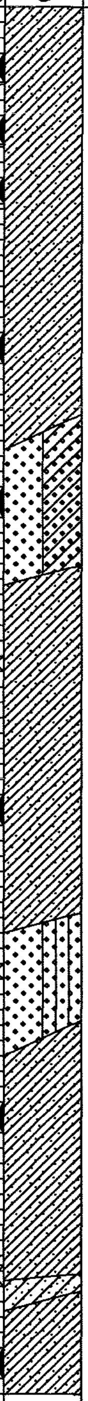
LOG OF BORING NO. TB-6

Permanente Creek Project
Mountain View and Los Altos, CA

PLATE 8

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Valencia St
Drill Rig Mobile B-53, 8" Hollow Stem
Top Elevation 113.1 Ft. **Date Drilled** 6/25/2009

Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Sample	Graphic	Soil Description
									Brown Sandy Lean CLAY (CL) Medium stiff to hard, dry
	27*	>4.5		10.1	108		⧓		
	31*	>4.5		10.1	108		⧓		
						5			
	17*	>4.5		11.7	103		⧓		
						10			
$TxUU_{(FM)}$= 4240 (1100)	13*			13.8	119		⧓		
						15			Brown Well Graded SAND with clay and gravel (SW-SC) Medium dense, moist, gravels to 2 inches
MA: -#4 = 61% -#200 = 8%	22*						⧓		
						20			Brown Sandy Lean CLAY (CL) Stiff to hard, moist, medium to coarse sand
	15						☒		
						25			
UC= 1460	13*	2.25		20.9	108		⧓		
						30			Brown Well Graded SAND with silt (SW-SM) Very dense, dry to moist, fine to coarse sand
	54						☒		
						35			Brown Sandy Lean CLAY (CL) Stiff to very stiff, moist, medium sand
	11*	2.25		20.6	102		⧓		
						40			
	14						☒		Brown Clayey SAND (SC) Medium dense, moist, medium sand
	14*	1.5		27.8	96		⧓		Brown Sandy Lean CLAY (CL) Stiff, moist, medium sand
						45			Bottom of Boring at 45 Feet

***Blows/foot converted to approximate standard penetration test blow counts.**

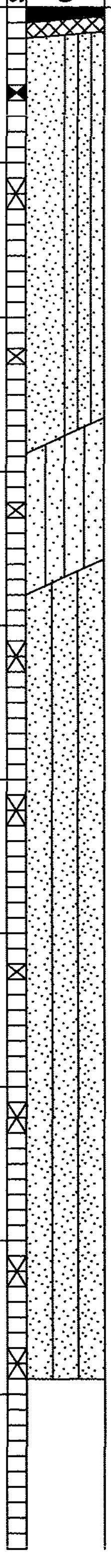
Santa Clara Valley Water District

LOG OF BORING NO. TB-7
Permanente Creek Project
Mountain View and Los Altos, CA

PLATE
9

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Eastwood Dr
Drill Rig Mobile B-53, 8" Hollow Stem
Top Elevation 186.9 Ft. **Date Drilled** 6/29/2009



Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							6 inches Asphalt Concrete
							6 inches Aggregate Base
-#200 = 3%	16*						Brown Poorly Graded SAND with silt and gravel (SP-SM) Medium dense dry to moist
						5	
MA: -#4 = 65% -#200 = 9%	26						
						10	
-#200 = 10%	20						
						15	Brown Sandy SILT (ML) Very stiff, dry, fine to medium sand
	28						
						20	Brown Silty SAND with gravel (SM) Medium dense to very dense, dry, fine to medium sand, gravels to 1-1/2 inches
-#200 = 27%	50						
						25	
-#200 = 18%	27						
						30	
	40						
						35	
MA: -#4 = 69% -#200 = 16%	42						
						40	
	60						
	56						
						45	Bottom of Boring at 45 Feet

***Blows/foot converted to approximate standard penetration test blow counts.**

Santa Clara Valley Water District

LOG OF BORING NO. TB-9

Permanente Creek Project
Mountain View and Los Altos, CA

PLATE **10**

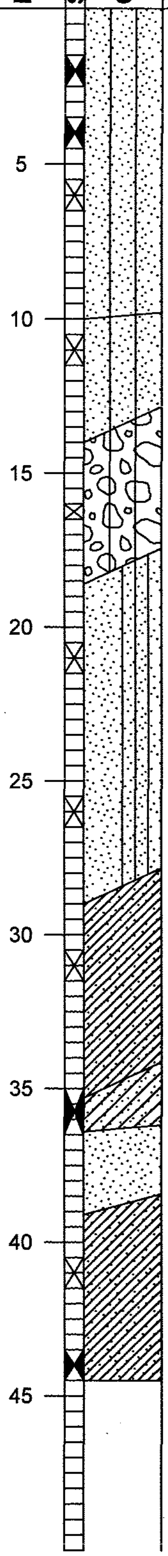
Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

LOG OF BORING 2009 PERMANENTE CREEK.GPJ SANTA CLARA VALLEY WATER.GDT 4/21/10

Boring Location Off Miramonte Ave
Drill Rig Marl M-57 Rhino, 6.25" Hollow Stem
Top Elevation 191.6 Ft. **Date Drilled** 7/1/2009

Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)
$\text{TxUU}_{(FM)}$= 3310 (220)	19*			11.6	114
MA: -#4 = 82% -#200 = 25%	11*				
	14				
MA: -#4 = 85% -#200 = 21%	10				
	16				
	32				
MA: -#4 = 61% -#200 = 11%	38				
	20	2			
UC= 180	53*			7.0	119
	20	2			
	25*	4		17.8	113

*Blows/foot converted to approximate standard penetration test blow counts.



Soil Description

Reddish Brown Silty SAND with gravel (SM) - Fill
Medium dense, dry, gravels to 3/4 inch

Reddish Brown Silty SAND with gravel (SM)
Loose, dry, fine to medium sand, gravels to 1/2 inch

Brown Silty GRAVEL with sand (GM)
Medium dense, dry, gravels to 1 inch

Brown Poorly Graded SAND with silt and gravel (SP-SM)
Medium dense, dry, medium to coarse sand, trace gravel to 3/4 inch

Brown Sandy Lean CLAY (CL)
Very stiff, moist, fine sand

Brown Clayey SAND (SC)
Medium dense to dense, moist, medium to coarse sand, gravel to 3/4 inch
Brown Poorly Graded SAND (SP)
Dense, moist, medium to coarse sand
Brown Lean CLAY with sand (CL)
Very stiff to hard, moist, fine sand

Medium to coarse sand
Bottom of Boring at 44.5 Feet

Santa Clara Valley Water District

LOG OF BORING NO. TB-10
Permanente Creek Project
Mountain View and Los Altos, CA

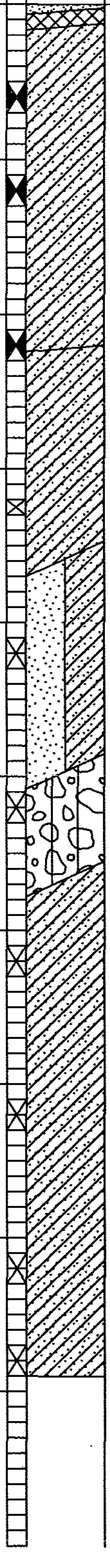
PLATE
11

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Off Miramonte Ave

Drill Rig Mobile B-53, 7" Hollow Stem

Top Elevation 194.0 Ft. **Date Drilled** 6/25/2009



Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							4 inches Portland Cement Concrete 6 inches Aggregate Base Brown Clayey SAND with gravel (SC) - Fill Very loose, moist, medium to coarse sand
-#200 = 34%	4 *	1.5		14.9	104		
						5	
TxUU$_{(FM)}$= 2120 (600)	9 *			12.9	118		
						10	
	28*			11.2	98		
							Brown Clayey SAND with gravel (SC) Medium dense, moist, gravels to 2 inches
						15	
MA: -#4 = 74% -#200 = 14%	10						
							Brown Poorly Graded SAND with clay and gravel (SP-SC) Medium dense, moist, gravel to 2 inches
						20	
-#200 = 10%	18						
						25	
	58						Brown Silty GRAVEL with sand (GM) Very dense, dry to moist, gravels to 2 inches
							Brown Clayey SAND (SC) Loose to very dense, moist, fine to medium sand, trace fine gravel
						30	
-#200 = 44%	6						
						35	
	51						With gravel at 36 feet
						40	
-#200 = 13%	35						
	26						
						45	Bottom of Boring at 44.5 Feet

***Blows/foot converted to approximate standard penetration test blow counts.**

Santa Clara Valley Water District

LOG OF BORING NO. TB-11

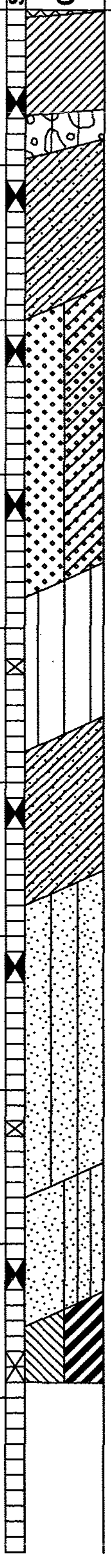
Permanente Creek Project
Mountain View and Los Altos, CA

PLATE
12

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Off Mountain View Ave
Drill Rig Mobile B-53, 7" Hollow Stem
Top Elevation 117.3 Ft. **Date Drilled** 6/24/2009

Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							3 inches of Aggregate Base Brown Sandy Lean CLAY (CL) - Fill Hard, dry to moist
	26*	>4.5		9.5	114		
							Brown Silty GRAVEL with sand (GM) Medium dense, dry to moist, gravels to 3/4 inch
						5	Brown Lean CLAY with sand (CL) Very stiff to hard, dry, medium to coarse sand
TxUU$_{(FM)}$= 2830 (600)	13*	>4.5		11.4	98		
						10	Brown Well Graded SAND with clay and gravel (SW-SC) Medium dense, dry, gravels to 2 inches
MA: -#4 = 66% -#200 = 6%	26*						
						15	
MA: -#4 = 56% -#200 = 5%	36*						
						20	Brown SILT with sand (ML) Stiff to very stiff, dry, fine sand
-#200 = 80%	15						
						25	Brown Sandy Lean CLAY (CL) Very stiff to hard, moist, medium sand
UC= 3040	21*	>4.5		20.5	108		
						30	Brown Silty SAND with gravel (SM) Dense, moist, medium to coarse sand, fine gravel
-#200 = 16%	36*			8.8	119		
						35	
	47						
						40	Brown Poorly Graded SAND with silt and gravel (SP-SM) Dense, moist, medium to coarse sand, fine gravel With some gravels at 40 feet
-#200 = 6%	37*						
							Brown Lean to Fat CLAY (CL-CH) Stiff to very stiff, moist
	16	1.5					
						45	Bottom of Boring at 44.5 Feet



***Blows/foot converted to approximate standard penetration test blow counts.**

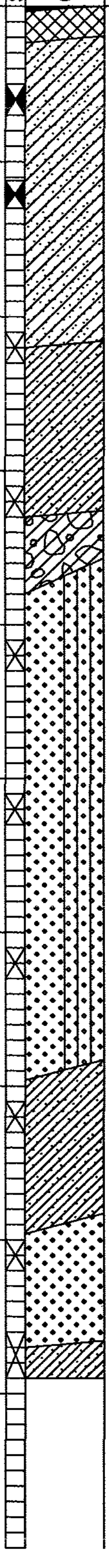
Santa Clara Valley Water District

LOG OF BORING NO. TB-12
Permanente Creek Project
Mountain View and Los Altos, CA

PLATE 13

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Arroyo Rd
Drill Rig Marl M-57 Rhino, 6.25" Hollow Stem
Top Elevation 124.3 Ft. **Date Drilled** 6/30/2009



Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							2 inches Asphalt Concrete 12 inches Aggregate Base Brown Clayey SAND with gravel (SC) Medium dense, dry, gravels to 1-1/2 inch, medium to coarse sand
$\text{TxUU}_{(FM)}$= 3720 (300)	17*			13.3	113		
						5	
LL= 40%; PI= 20%	14*			11.9	105		
						10	
	25						Brown Sandy Lean CLAY (CL) Very stiff, dry, fine to medium sand
						15	
-#200 = 65%	32	4					
							Brown Clayey GRAVEL (GC) Dense, moist, gravel to 3/4 inch
							Brown Well Graded SAND with silt and gravel (SW-SM) Dense to very dense, dry to moist, medium to coarse sand, gravels to 1/2 inch
						20	
MA: -#4 = 80% -#200 = 10%	49						
						25	
	56						
						30	
MA: -#4 = 76% -#200 = 8%	61						
						35	Brown Sandy Lean CLAY (CL) Stiff, moist, fine to medium sand
	13	1					
-#200 = 4%	98					40	Brown Well Graded SAND with gravel (SW) Very dense, moist, medium to coarse sand, fine gravel
	15	2					Brown Sandy Lean CLAY (CL) Stiff to very stiff, moist, medium sand Bottom of Boring at 44.5 Feet
						45	

*Blows/foot converted to approximate standard penetration test blow counts.

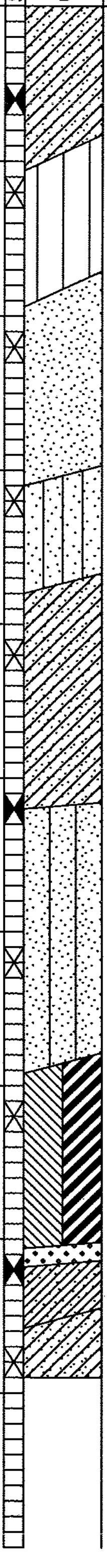
Santa Clara Valley Water District

LOG OF BORING NO. TB-13
Permanente Creek Project
Mountain View and Los Altos, CA

PLATE 14

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

Boring Location Off Miramonte Ave
Drill Rig Marl M-57 Rhino, 6.25" Hollow Stem
Top Elevation 190.6 Ft. **Date Drilled** 7/1/2009



Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	Soil Description
							Brown Clayey SAND with gravel (SC) - Fill Medium dense, dry to moist, medium to coarse sand, gravel to 1/2 inch
$\text{TxUU}_{(FM)}$= 2750 (300)	17*			10.6	120		
						5	
MA: -#4 = 99% -#200 = 73%	4						Brown SILT with sand (ML) Soft to medium stiff, moist
						10	
MA: -#4 = 95% -#200 = 5%	4						Brown Poorly Graded SAND (SP) Loose, dry to moist, fine to medium sand
						15	
-#200 = 66%	5						Brown Sandy SILT (ML) Medium stiff, moist, fine sand
							Brown Clayey SAND (SC) Medium dense, moist, fine to medium sand
						20	
-#200 = 45%	20						
						25	
	15*	4		21.9	106		Brown Silty SAND (SM) Loose to medium dense, moist, fine sand
						30	
MA: -#4 = 91% -#200 = 30%	9						
						35	Brown Lean to Fat CLAY (CL-CH) Stiff to very stiff, moist, medium plasticity
	28	1.75					
						40	
	13*	2.5		18.8	112		Brown Well Graded SAND (SW) Medium dense, moist, medium to coarse sand Brown Sandy Lean CLAY (CL) Stiff to very stiff, moist, medium to coarse sand
	38						Brown Clayey SAND with gravel (SC) Dense, moist, medium to coarse sand, gravels to 2 inches
						45	Bottom of Boring at 44.5 Feet

*Blows/foot converted to approximate standard penetration test blow counts.

Santa Clara Valley Water District

LOG OF BORING NO. TB-14

Permanente Creek Project
Mountain View and Los Altos, CA

PLATE 15

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

LOG OF BORING 2009 PERMANENTE CREEK.GPJ SANTA CLARA VALLEY WATER.GDT 4/21/10

MAJOR DIVISIONS					TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	**GRAVELS** MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	●	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	○	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	○	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	▨	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	⁙	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	⁚	POORLY GRADED SANDS WITH OR WITHOUT GRAVELS, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	⁝	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	▨	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	**SILTS AND CLAYS** LIQUID LIMIT 50% OR LESS		ML	▥	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	▨	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	▤	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	▥	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	▨	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	≈	ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	⁂	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION - ASTM D2487 - 00

R-Value — R-Value Number
Perm — Permeability
Consol — Consolidation
LL — Liquid Limit (%)
PI — Plastic Index (%)
G_s — Specific Gravity
MA — Particle Size Analysis
■ — Shelby Tube Sample
⊠ — SPT Sample
⧓ — Modified California Sample
▽ — Groundwater Level First Encountered (Date Measured)
▼ — Groundwater Level (Date Measured)

	Shear Strength (psf)	Confining Pressure	
TxUU (FM) or (S)	3200	(2600)	— Unconsolidated Undrained Triaxial Shear (field moisture or saturated)
TxCU (P)	3200	(2600)	— Consolidated Undrained Triaxial Shear (with or without pore pressure measurement)
TxCD	3200	(2600)	— Consolidated Drained Triaxial Shear
SSCU (P)	3200	(2600)	— Simple Shear Consoildated Undrained (with or without pore pressure measurement)
SSCD	3200	(2600)	— Simple Shear Consoildated Drained
DSCD	2700	(2000)	— Consolidated Drained Direct Shear
DSCU	2000	(1000)	— Consoilidated Undrained Direct Shear
UC	470		— Unconfined Compressive Shear
LVS	700		— Laboratory Vane Shear

KEY TO TEST DATA

Santa Clara Valley Water District

Soil Classification and Test Data Key

Permanente Creek Project
Mountain View and Los Altos, CA

PLATE
16

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	26244001				

SOIL CLASSIFICATION 2009 PERMANENTE CREEK.GPJ SANTA CLARA VALLEY WATER.GDT 4/22/10

APPENDIX A

GREGG

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

June 25, 2009

Santa Clara Valley Water District
Attn: Steen Wu

Subject: CPT Site Investigation
Permanente Creek
Mt. View, California
GREGG Project Number: 09-098MA

Dear Mr. Wu:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVOST Laser Induced Fluorescence	(UVOST)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☐
10	SPT Energy Calibration	(SPTE)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (Feet)	Depth of Groundwater Samples (Feet)	Depth of Soil Samples (Feet)	Depth of Pore Pressure Dissipation Tests (Feet)
CPT-01	6/23/09	75	-	-	62.5
CPT-02	6/23/09	75	-	-	-
CPT-03	6/23/09	75	-	-	-
CPT-04	6/23/09	75	-	-	-
CPT-05	6/24/09	45	-	-	-
CPT-06	6/24/09	75	-	-	70.9
CPT-07	6/24/09	45	-	-	-

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Campanella, R.G. and I. Weemees, "Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies", Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, "Reliability of Soil Gas Sampling and Characterization Techniques", International Site Characterization Conference - Atlanta, 1998.

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Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers" BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure

(CPT)

Gregg Drilling & Testing, Inc. carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, Figure CPT. The soundings are conducted using a 20 ton capacity cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and dynamic pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

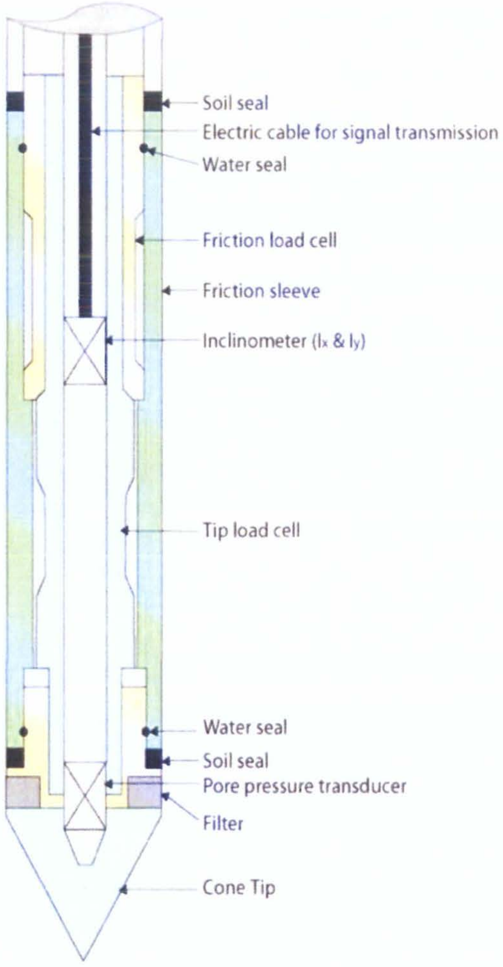


Figure CPT

The cone also contains a porous filter element located directly behind the cone tip (u_2), Figure CPT. It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain dynamic pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

When the soundings are complete, the test holes are grouted using a Gregg Drilling support rig. The grouting procedure consists of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

Cone Penetration Test Data & Interpretation

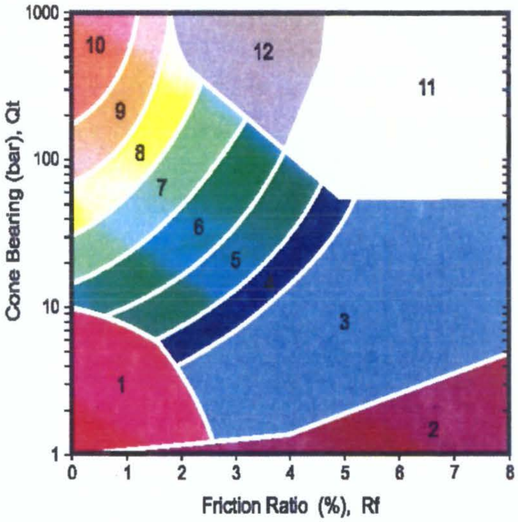
The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg InSitu and Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

(After Robertson, et al., 1986)



ZONE	SBT	
1	🟥	Sensitive, fine grained
2	🟪	Organic materials
3	🟦	Clay
4	🟦	Silty clay to clay
5	🟩	Clayey silt to silty clay
6	🟩	Sandy silt to clayey silt
7	⬜	Silty sand to sandy silt
8	🟨	Sand to silty sand
9	🟧	Sand
10	🟥	Gravely sand to sand
11	⬜	Very stiff fine grained*
12	⬜	Sand to clayey sand*

*over consolidated or cemented

Figure SBT

Cone Penetration Test (CPT) Interpretation

Gregg have recently updated their CPT interpretation and plotting software (2007). The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1 Units for display (Imperial or metric) (atm. pressure, $p_a = 0.96$ tsf or 0.1 MPa)
2 Depth interval to average results,(ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3 Elevation of ground surface (ft or m)
4 Depth to water table, z_w (ft or m) – input required
5 Net area ratio for cone, a (default to 0.80)
6 Relative Density constant, C_{Dr} (default to 350)
7 Young's modulus number for sands, α (default to 5)
8 Small strain shear modulus number
  a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
  b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9 Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10 Over Consolidation ratio number, k_{ocr} (default to 0.3)
11 Unit weight of water, (default to $\gamma_w = 62.4$ lb/ft^3 or 9.81 kN/m^3)

Column

1	Depth, z, (m) – CPT data is collected in meters	
2	Depth (ft)	
3	Cone resistance, q_c (tsf or MPa)	
4	Sleeve friction, f_s (tsf or MPa)	
5	Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)	
6	Other – any additional data, if collected, e.g. electrical resistivity or UVIF	
7	Total cone resistance, q_t (tsf or MPa)	$q_t = q_c + u(1-a)$
8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note

11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \gamma z$
12	Insitu pore pressure, u_o (tsf)	$u_o = \gamma_w (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u - u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with Ic)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT listed below — Lunne et al. (1997)

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n — Lunne et al. (1997)

4 SBT_n Index, I_c — $I_c = ((3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa)\,(pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, $n = 0.5$ (clean sand)
When $I_c > 3.30$, $n = 1.0$ (clays)
When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} (based on Normalized SBT_n) (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60} C_N.$
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, ϕ', (degrees) $\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go
a. $G_o = S_G(q_t\ \sigma'_{vo}\ pa)^{1/3}$ *For SBT_n 5, 6, 7*
b. $G_o = C_G\, q_t$ *For SBT_n 1, 2, 3& 4*
Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $\mathrm{OCR} = k_{ocr} Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

SBT Zones **SBT_n Zones**

The following updated and simplified SBT descriptions have been used in the software:

1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		
7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

* heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print 'clays & silty clays')

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

Pore Pressure Dissipation Tests
(PPDT)

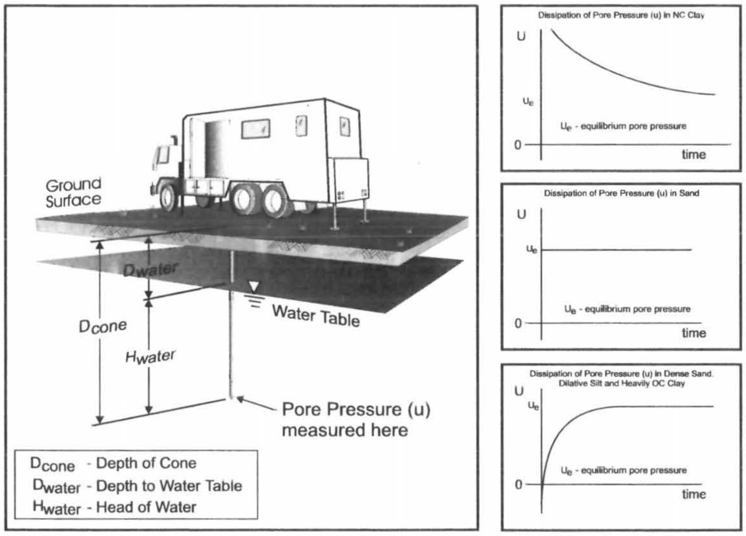
Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals measured hydrostatic water pressures and determined the approximate depth of the ground water table. A PPDT is conducted when the cone is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u*) with time is measured behind the tip of the cone and recorded by a computer system.

Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1991.


Ground Surface
Dwater
Dcone
Hwater
Water Table
Pore Pressure (u) measured here
Dcone - Depth of Cone
Dwater - Depth to Water Table
Hwater - Head of Water
Dissipation of Pore Pressure (u) in NC Clay
U
Ue
Ue - equilibrium pore pressure
0
time
Dissipation of Pore Pressure (u) in Sand
U
Ue
Ue - equilibrium pore pressure
0
time
Dissipation of Pore Pressure (u) in Dense Sand, Dilative Silt and Heavily OC Clay
U
Ue
Ue - equilibrium pore pressure
0
time

Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors:
1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

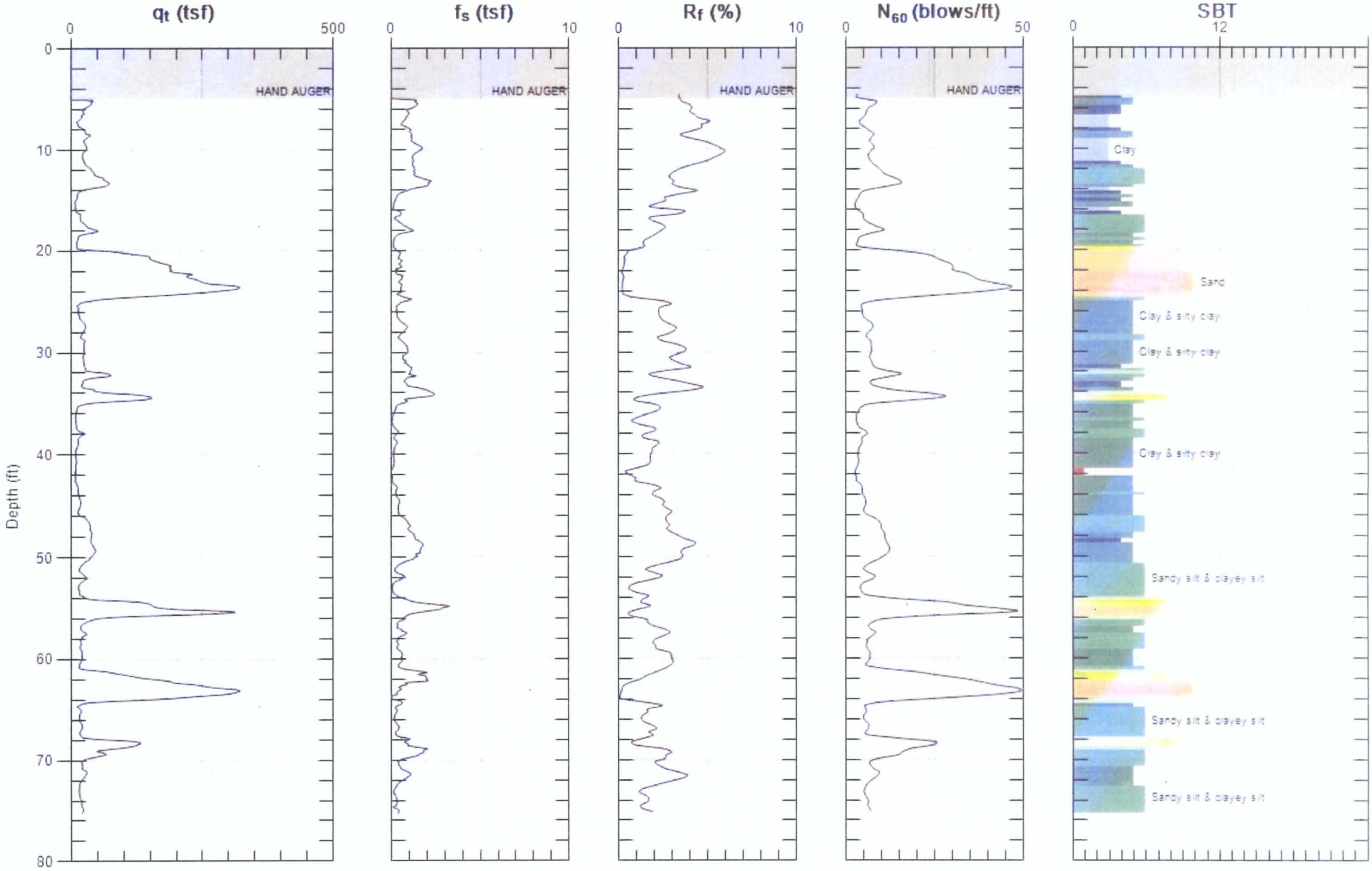
Figure PPDT

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-01

Engineer: S.WU
Date: 6/23/2009 08:31



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

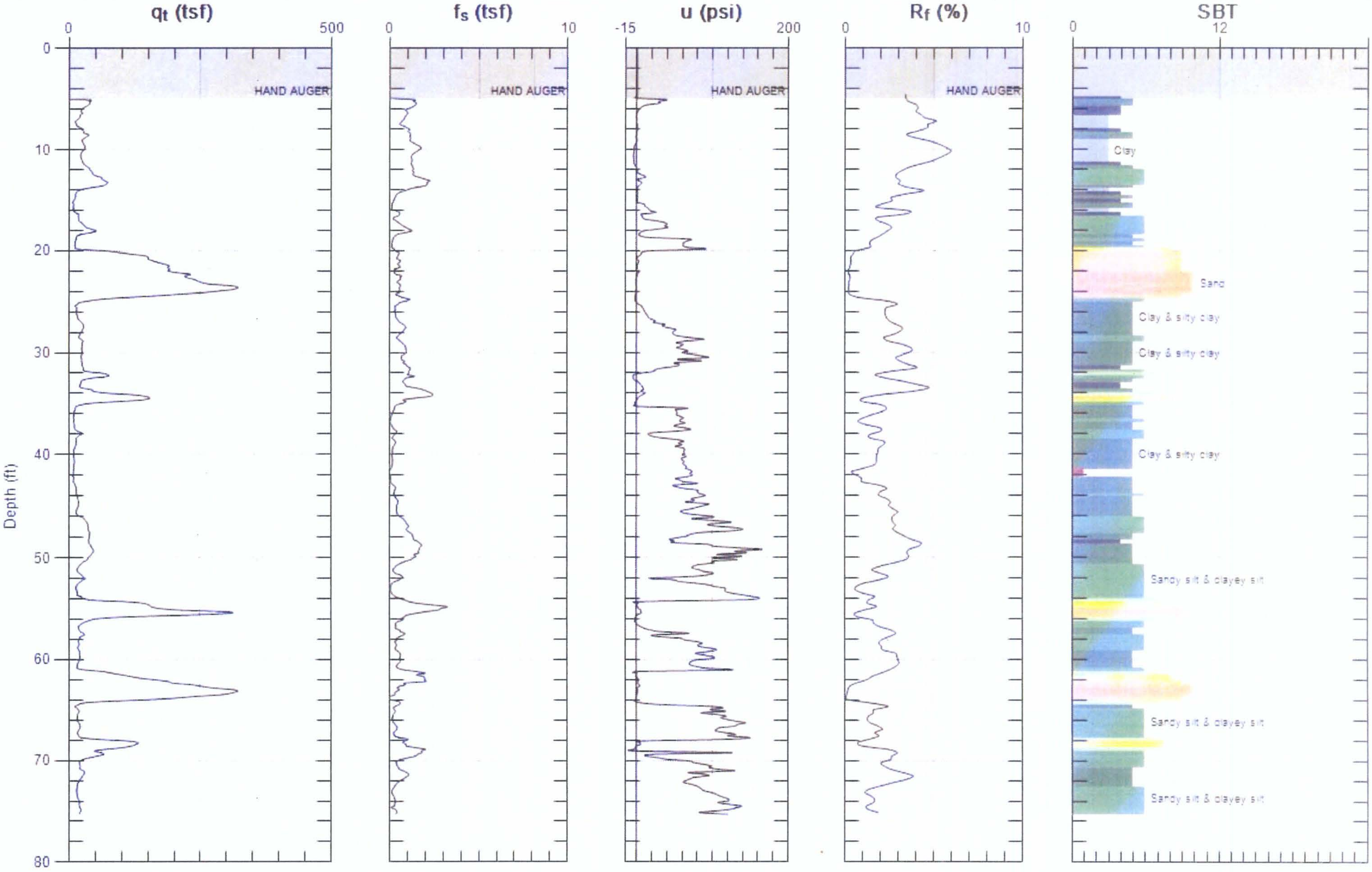
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-01

Engineer: S.WU
Date: 6/23/2009 08:31



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

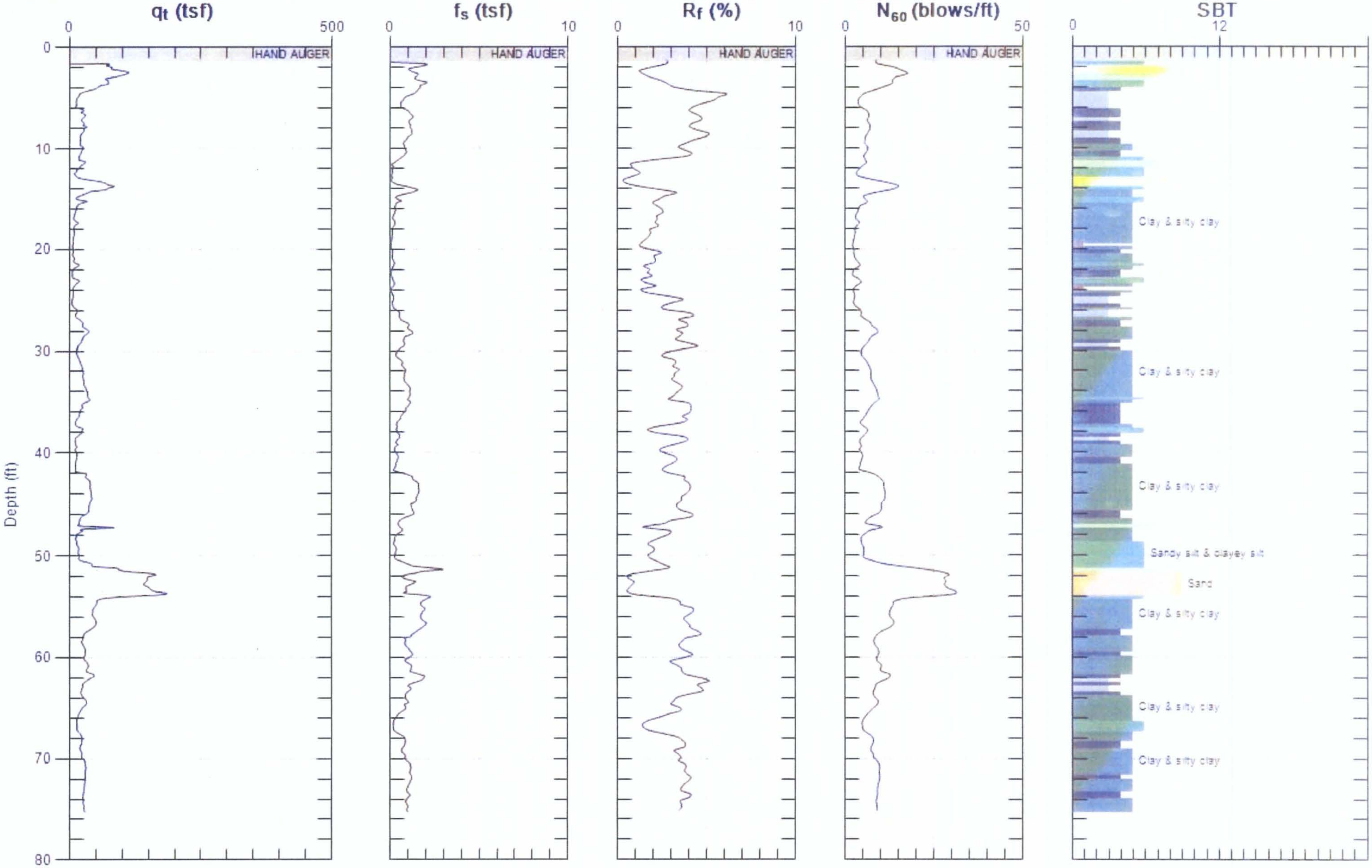
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-02

Engineer: S.WU
Date: 6/23/2009 10:53



Max Depth: 75.295 (ft)
Avg Interval: 0.328 (ft)

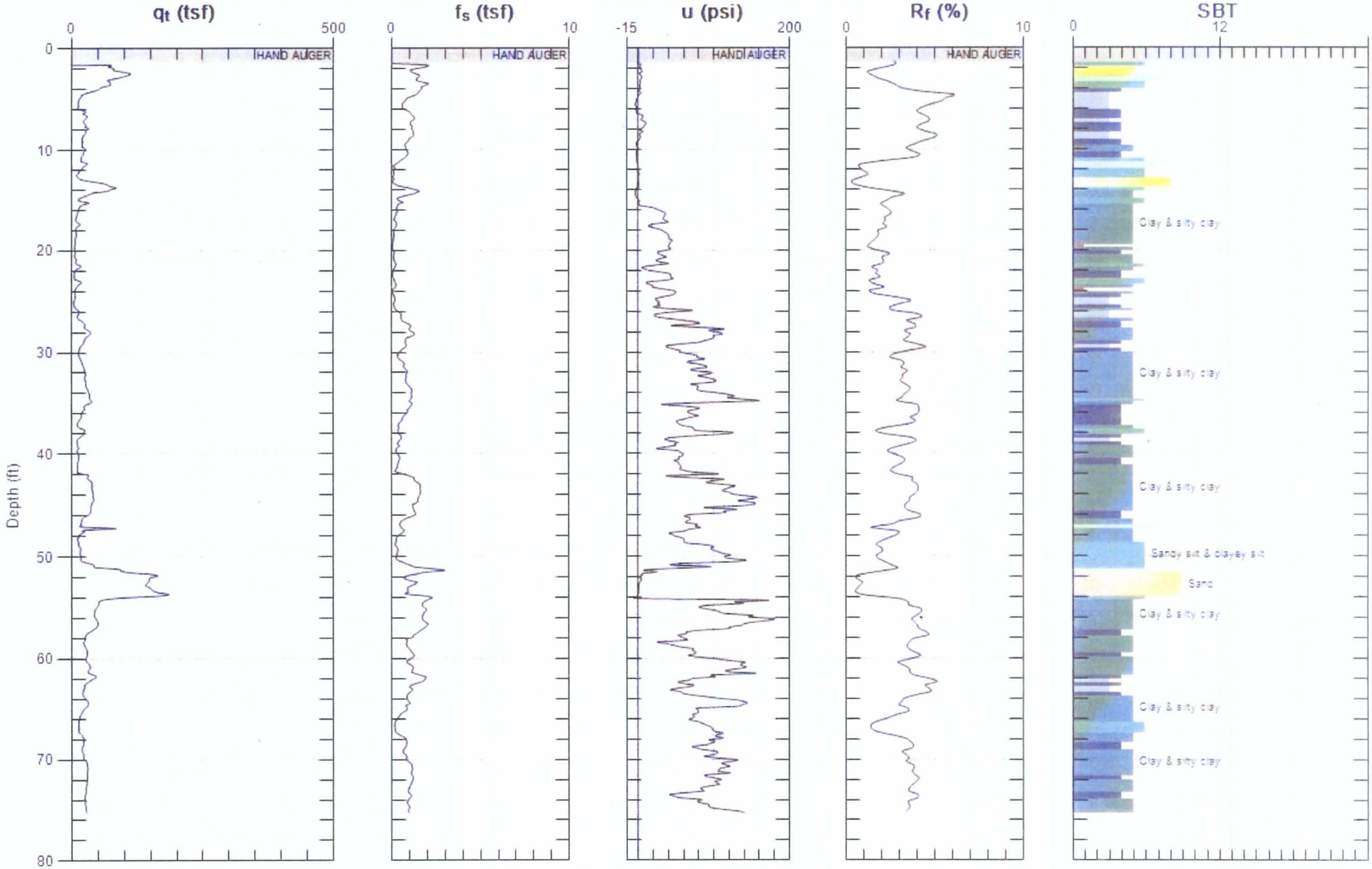
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-02

Engineer: S.WU
Date: 6/23/2009 10:53



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

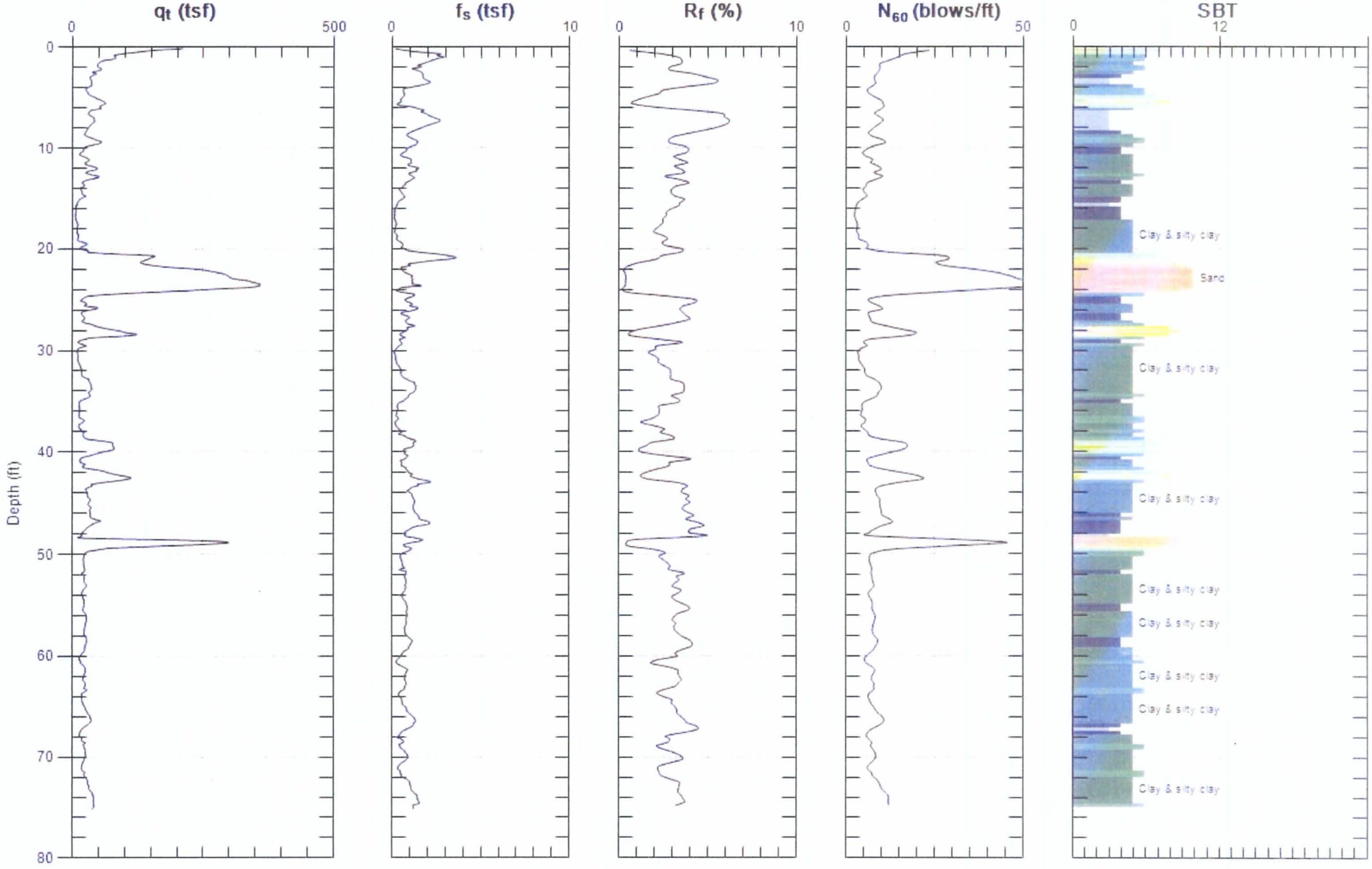
SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK

Sounding: CPT-03

Engineer: S.WU

Date: 6/23/2009 11:49



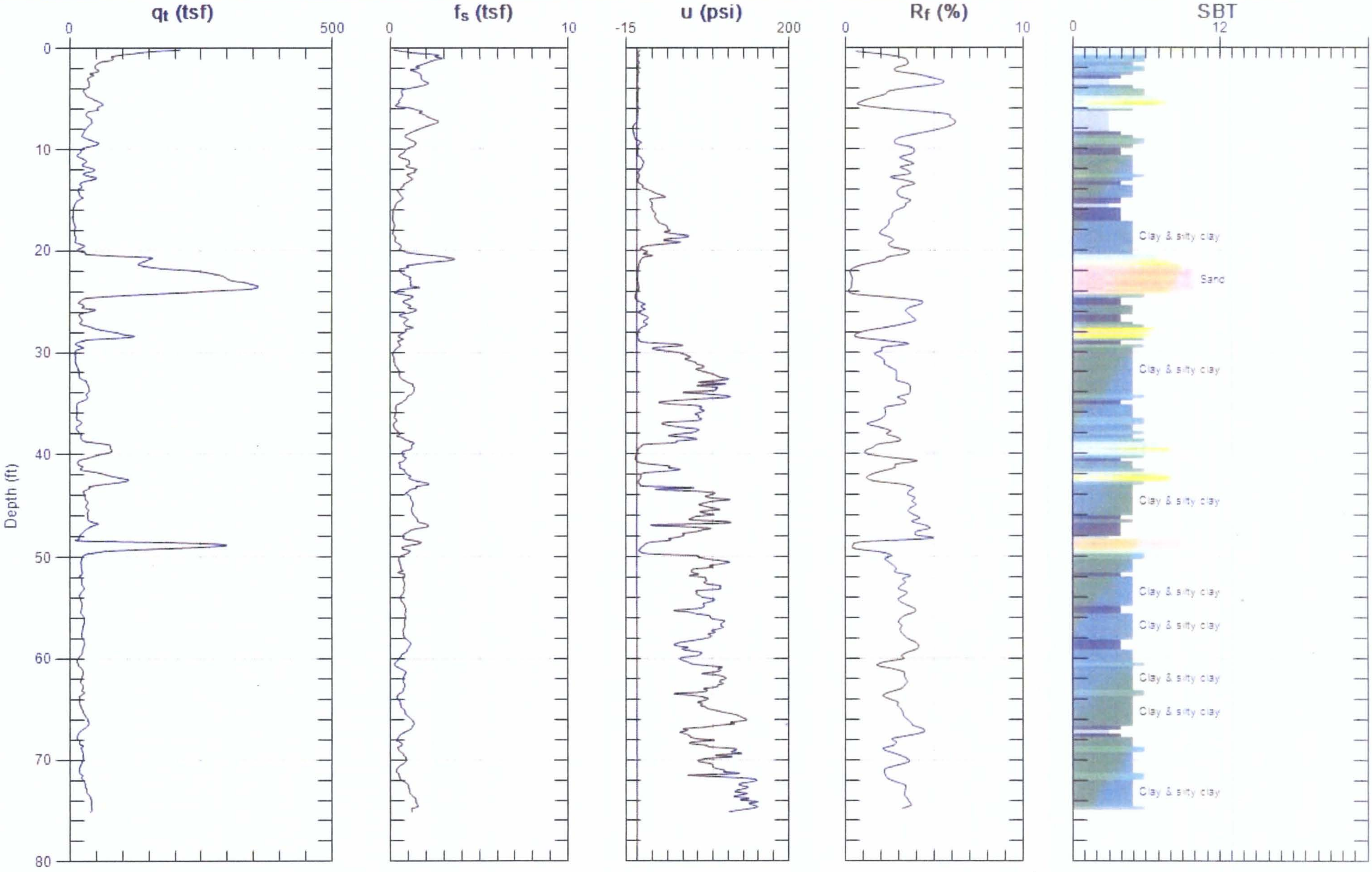
Max. Depth: 75.131 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-03
Engineer: S.WU
Date: 6/23/2009 11:49

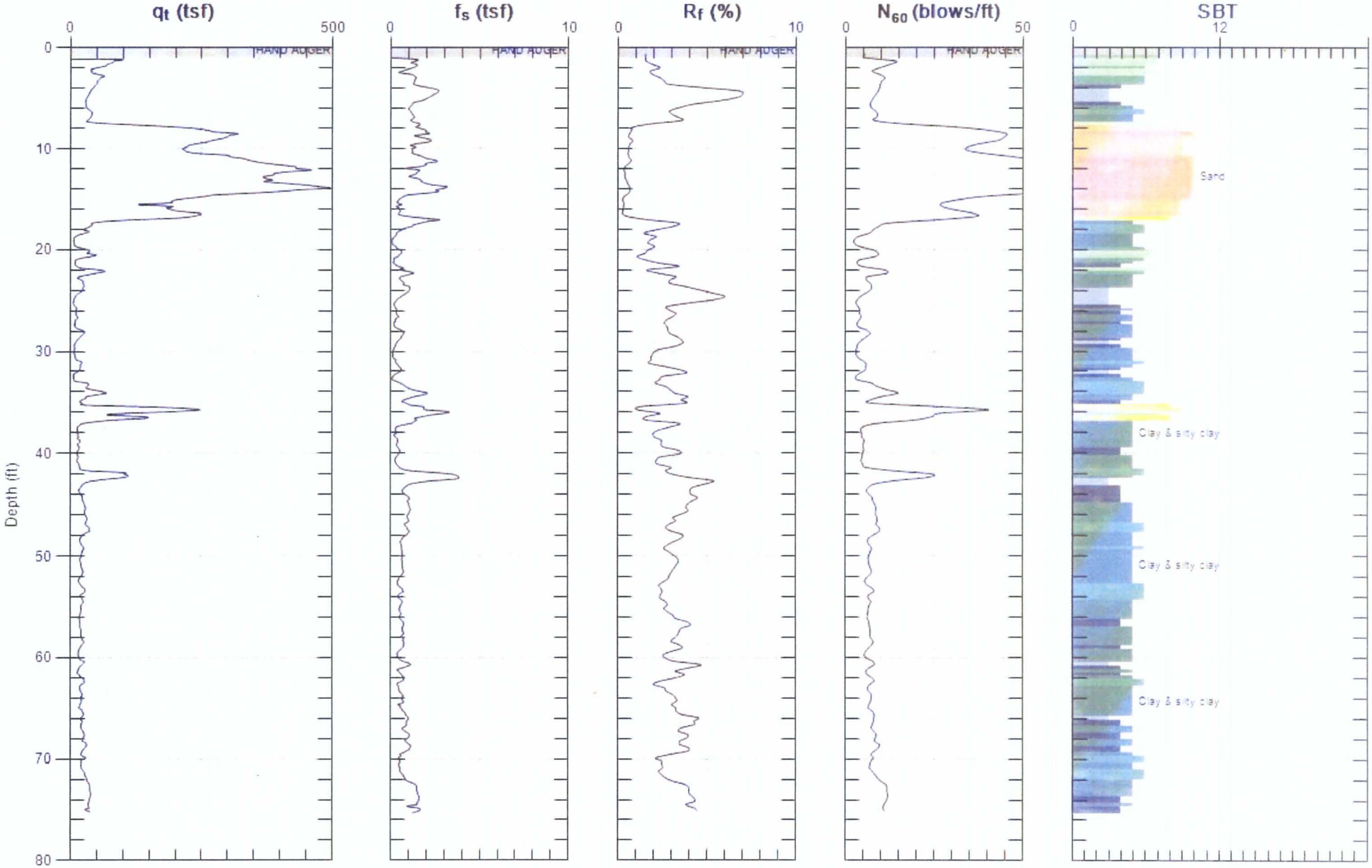


Max. Depth: 75.131 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-04
Engineer: S.WU
Date: 6/23/2009 02:13

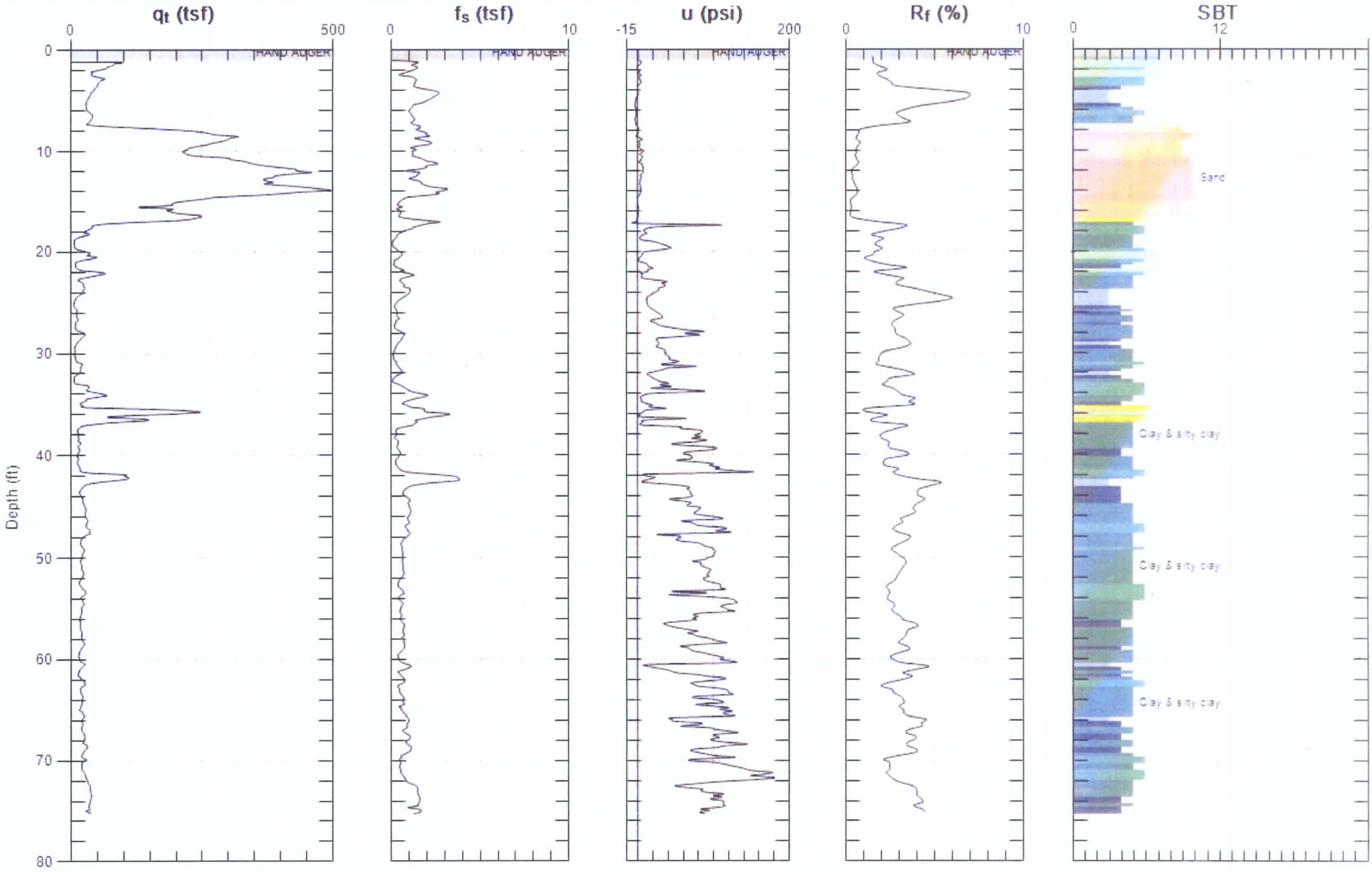


Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-04
Engineer: S.WU
Date: 6/23/2009 02:13



Max. Depth: 75.295 (ft)
Avg. Interval: 0.328 (ft)

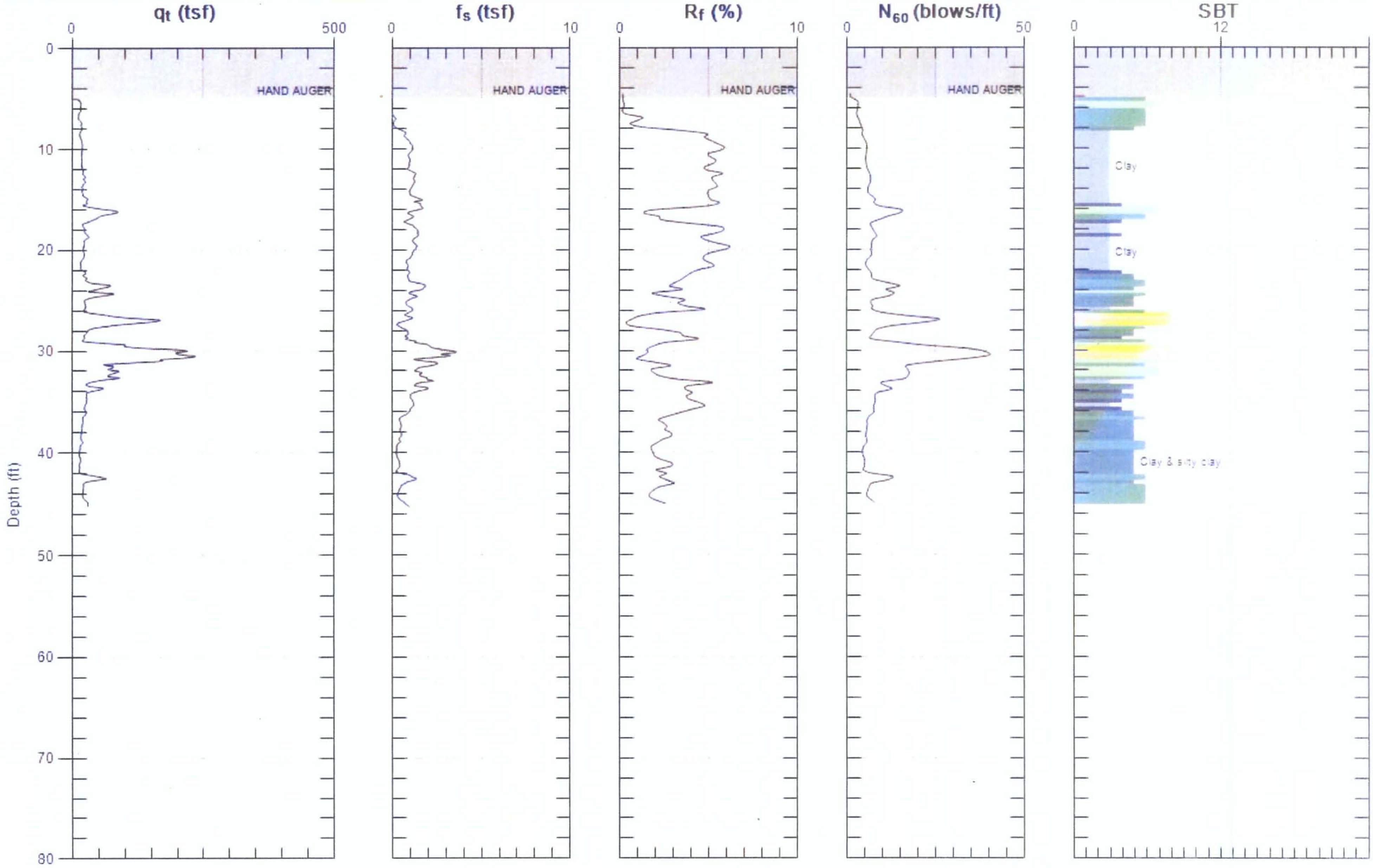
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-05

Engineer: S.WU
Date: 6/24/2009 08:05

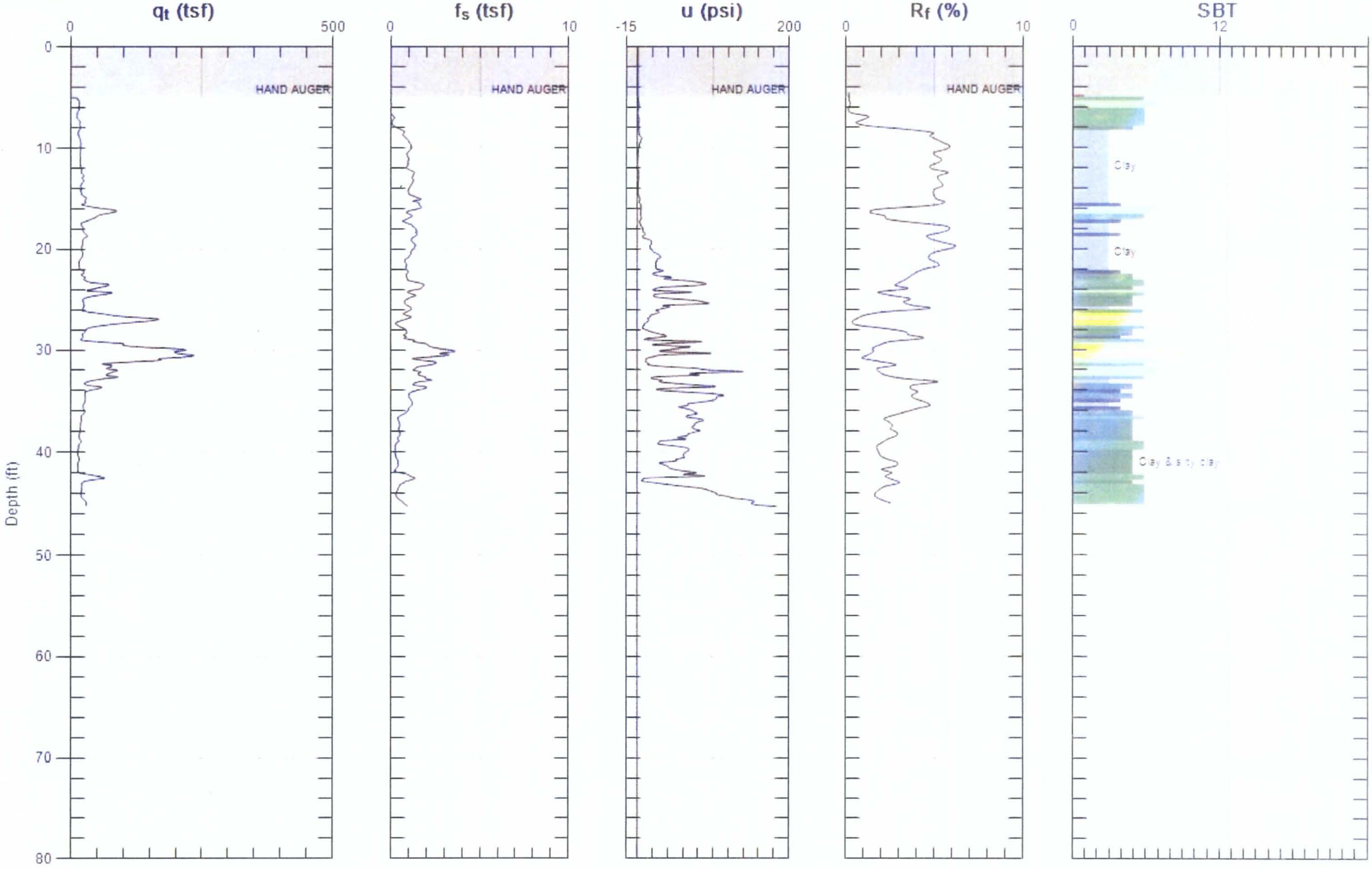


Max. Depth: 45.276 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-05
Engineer: S.WU
Date: 6/24/2009 08:05



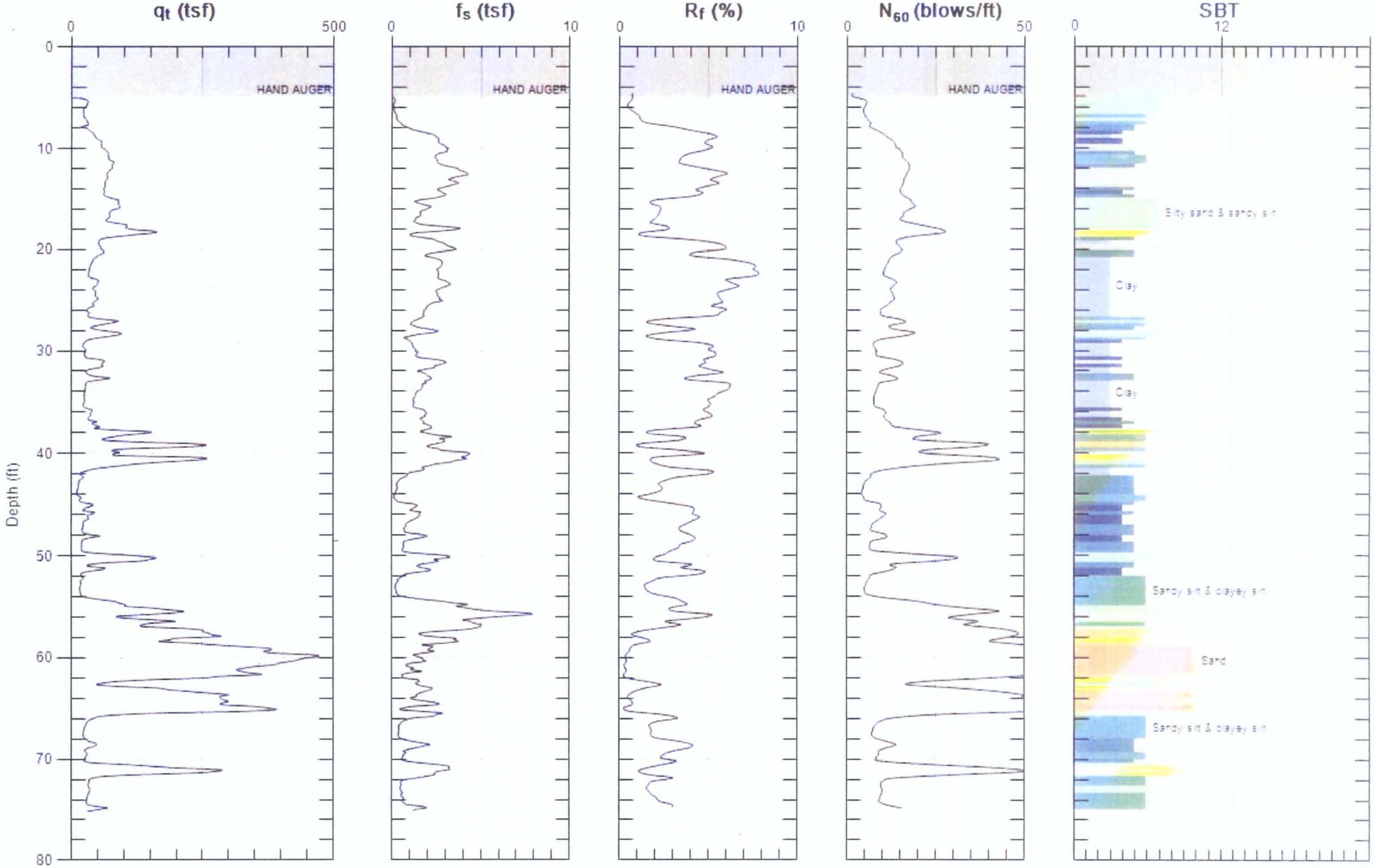
Max. Depth: 45.276 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-06

Engineer: S.WU
Date: 6/24/2009 09:18



Max. Depth: 75.131 (ft)
Avg. Interval: 0.328 (ft)

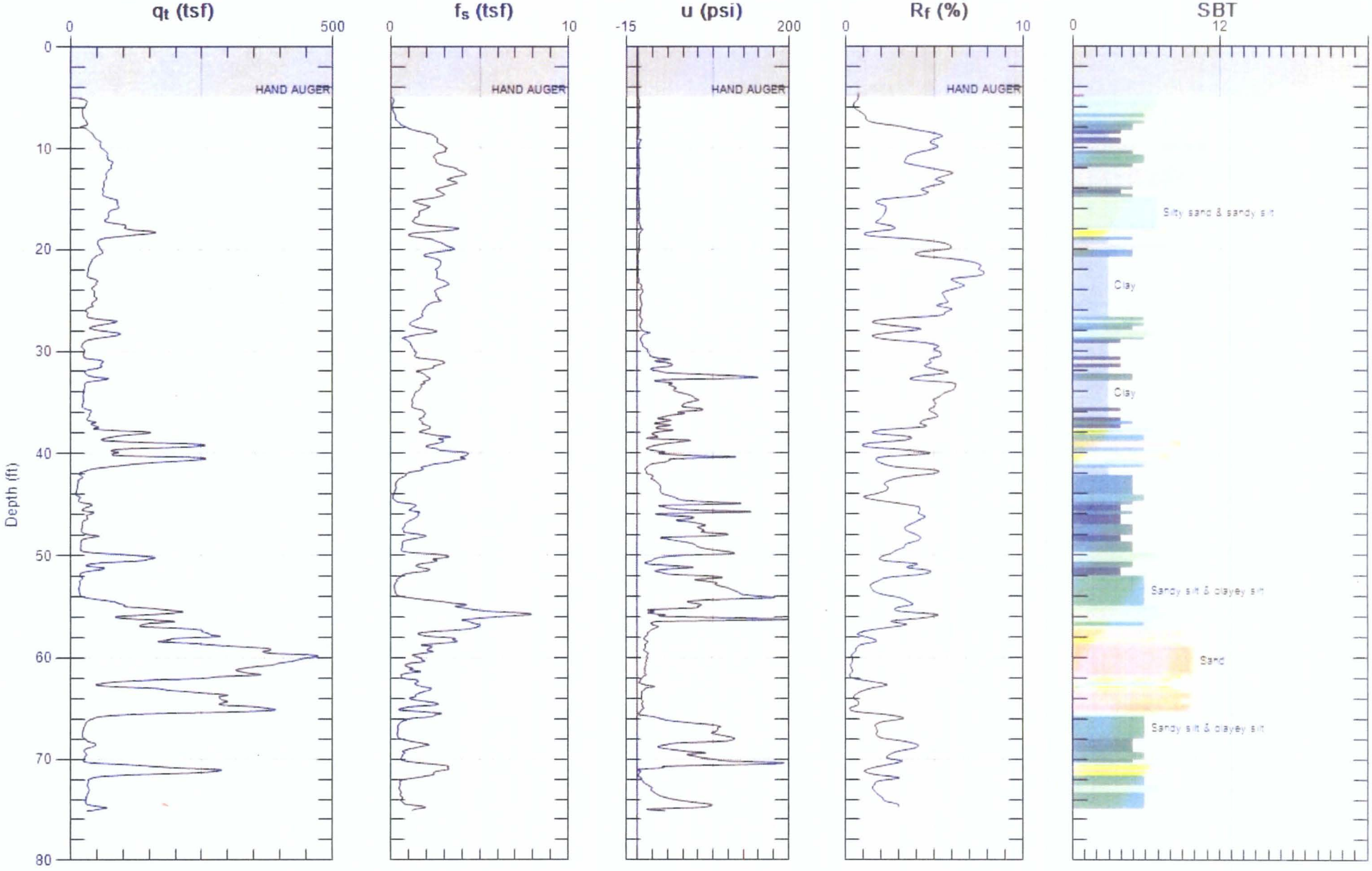
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-06

Engineer: S.WU
Date: 6/24/2009 09:18



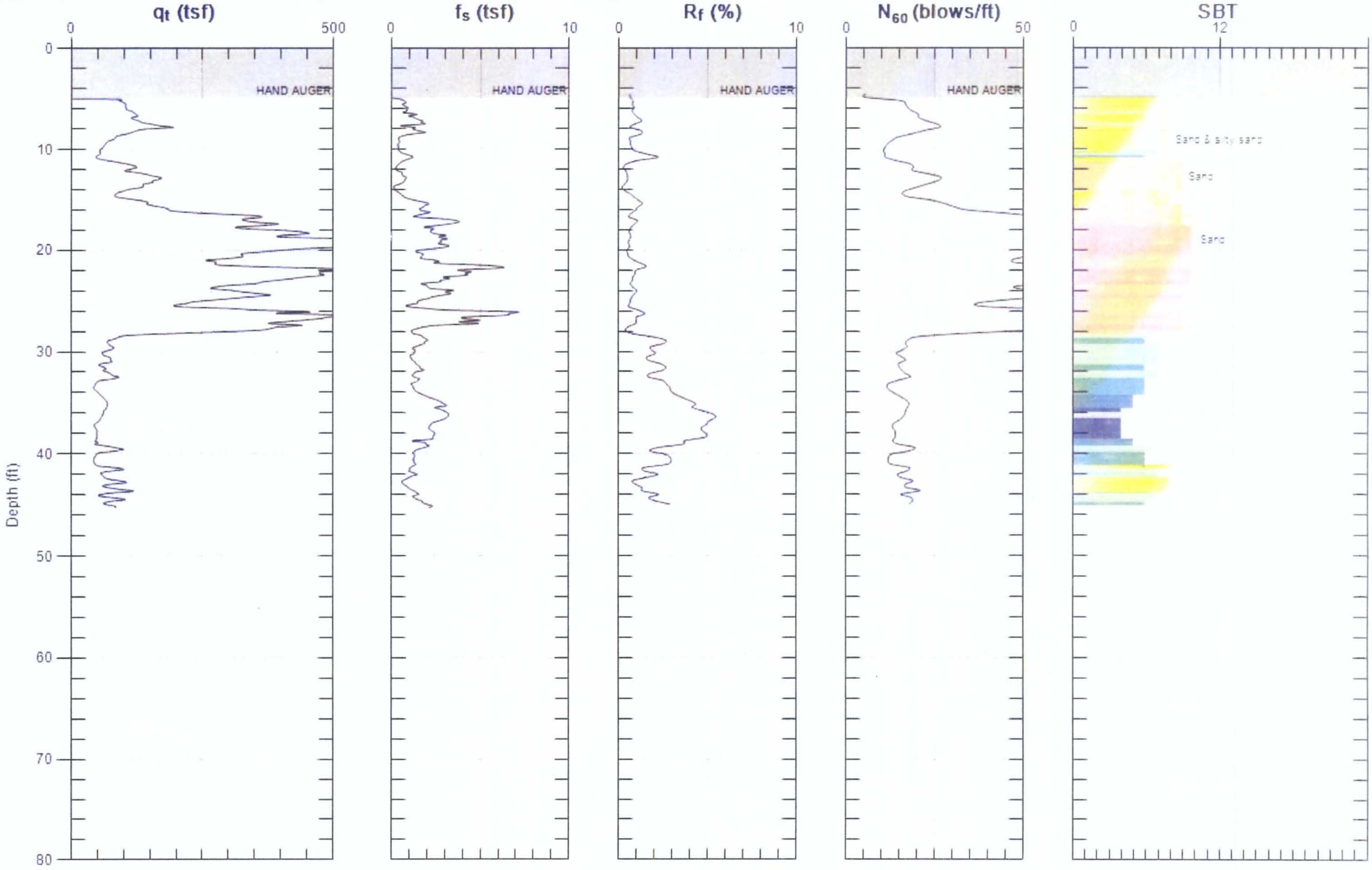
Max Depth: 75.131 (ft)
Avg Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-07
Engineer: S.WU
Date: 6/24/2009 01:39



Max Depth: 45.276 (ft)
Avg Interval: 0.328 (ft)

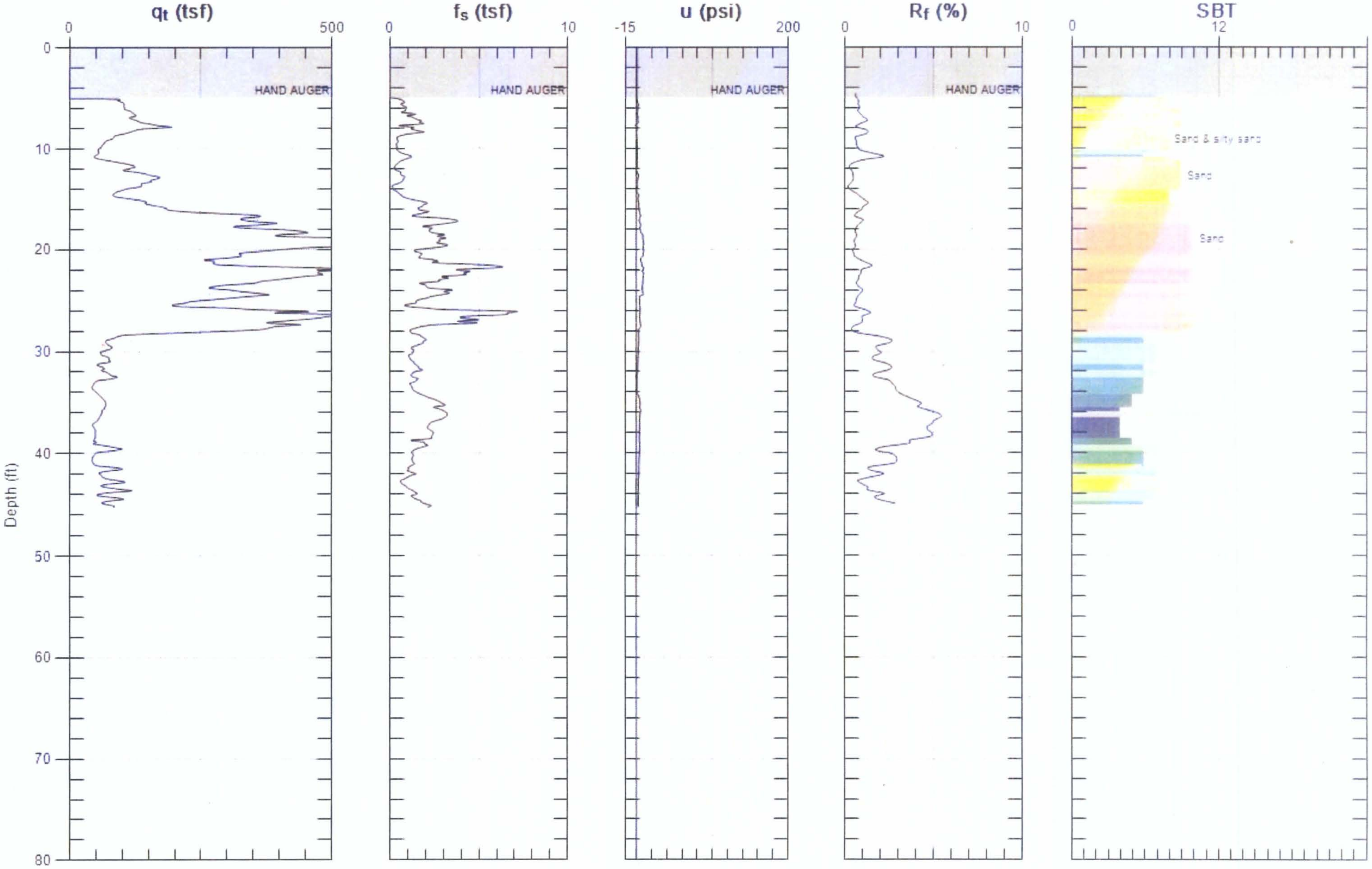
SBT: Soil Behavior Type (Robertson 1990)

GREGG

SANTA CLARA WATER DIST.

Site: PERMANENTE CREEK
Sounding: CPT-07

Engineer: S.WU
Date: 6/24/2009 01:39



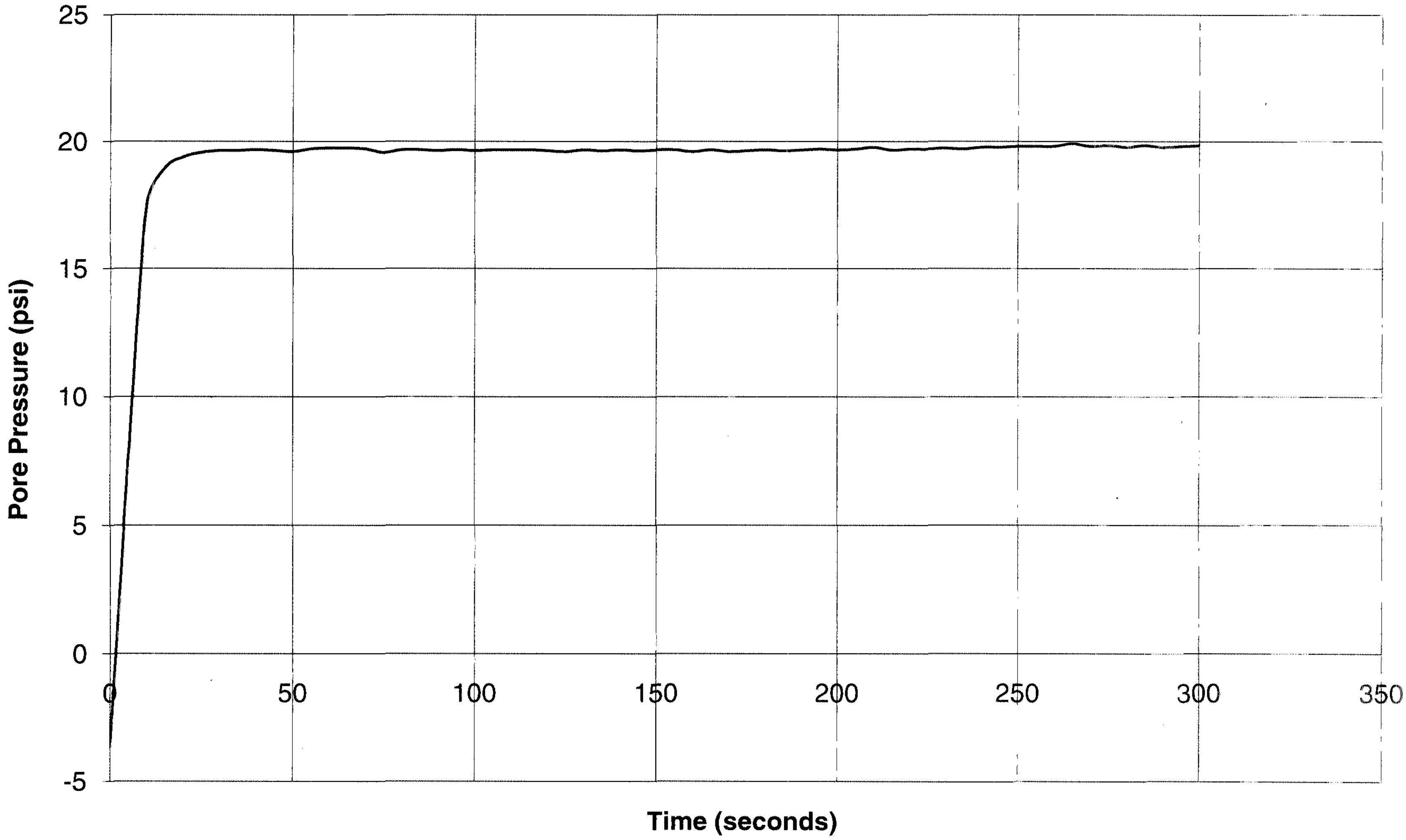
Max. Depth: 45.276 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

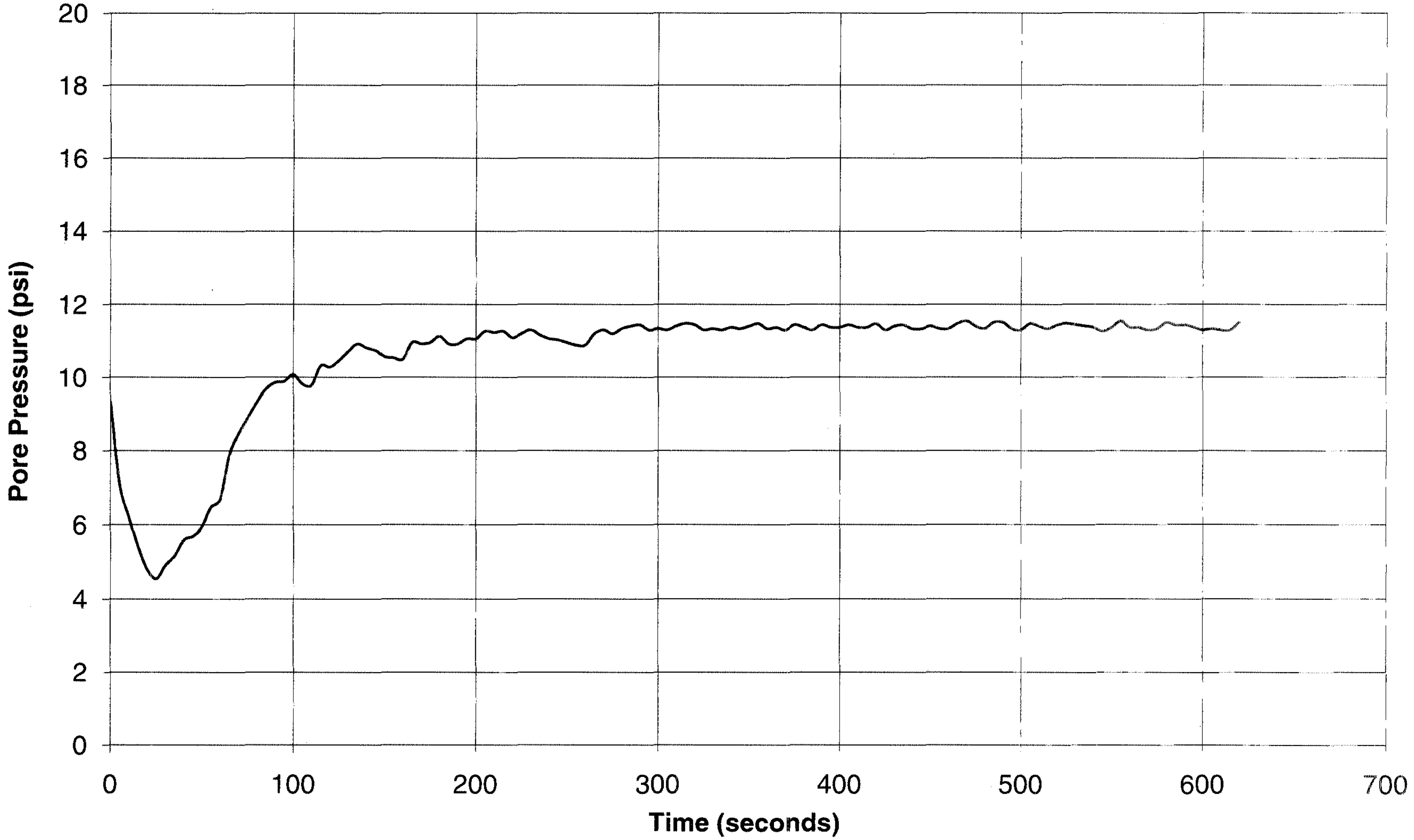
Sounding: CPT-01
Depth: 62.5
Site: PERMANENTE CREEK
Engineer: S.WU



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-06
Depth: 70.866
Site: PERMANENTE CREEK
Engineer: S.WU



APPENDIX B

Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D 2937)

Job No: 204-013a **Date:** 08/18/09
Client: SCVWD **By:** RU
Project: Permanente Creek, Mountain View - 10244001

Boring:	TB-1	TB-1	TB-1	TB-1	TB-1	TB-2	TB-2	TB-2
Sample:								
Depth, ft:	6.0	10.5	31.0	41.0	44.0	36.0	41.0	44.0
Visual Description:	Black Sandy Lean CLAY	Gray Clayey SAND	Brown Sandy Lean CLAY	Brown CLAY w/ Sand	Brown CLAY w/ Sand	Brown Lean Clayey SAND	Gray Sandy CLAY near Clayey SAND (Silty)	Olive Gray Sandy CLAY near Clayey SAND
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Total Vol cc	96.0	92.0	96.0	97.4	96.0	151.6	157.0	154.9
Vol Solids,cc	59.6	56.0	57.0	56.4	59.6	93.2	92.7	94.2
Vol Voids,cc	36.4	36.0	39.0	41.0	36.4	58.4	64.2	60.7
Moisture, %	18.9	17.9	25.2	26.6	22.6	23.0	25.6	23.8
Wet Unit wt, pcf	124.5	121.1	125.4	123.7	128.4	127.5	125.2	127.0
Dry Unit wt, pcf	104.7	102.7	100.2	97.7	104.7	103.7	99.6	102.5
Saturation, %	83.4	75.3	99.5	99.0	100.0	99.1	99.9	99.8
Porosity, %	37.9	39.1	40.6	42.1	37.9	38.5	40.9	39.2
Air filled Poros.,%	6.3	9.6	0.2	0.4	0.0	0.4	0.0	0.1
Water filled Poros.,%	31.6	29.5	40.4	41.7	37.9	38.2	40.9	39.1
Void Ratio	0.61	0.64	0.68	0.73	0.61	0.63	0.69	0.64
Series	1	2	3	4	5	6	7	8

Note: If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

COOPER
TESTING LABORATORY

Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D 2937)

Job No:	204-013b	**Date:**	08/18/09
Client:	SCVWD	**By:**	RU
Project:	Permanente Creek, Mountain View - 10244001		

Boring:	TB-3	TB-4	TB-4	TB-5	TB-5	TB-5	TB-6	TB-6
Sample:								
Depth, ft:	6.0	35.5	44.0	15.5	26.0	44.0	11.0	26.0
Visual Description:	Brown Silty SAND w/ Gravel	Brown Clayey SAND, trace Gravel	Gray Clayey SAND, trace Gravel	Brown Clayey SAND	Brown Clayey SAND, trace Gravel	Brown Sandy CLAY (Silty)	Reddish Brown Sandy CLAY	Reddish Brown Sandy SILT (slightly plastic)
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Total Vol cc	404.0	156.4	156.4	445.1	149.9	154.3	150.4	152.2
Vol Solids,cc	251.6	96.3	90.9	299.1	96.7	87.5	94.3	83.1
Vol Voids,cc	152.4	60.1	65.6	146.0	53.2	66.7	56.1	69.1
Moisture, %	6.2	23.0	26.5	13.1	16.9	28.0	10.4	28.4
Wet Unit wt, pcf	111.5	127.7	123.9	128.2	127.2	122.5	116.8	118.2
Dry Unit wt, pcf	105.1	103.9	98.0	113.4	108.9	95.7	105.8	92.1
Saturation, %	27.5	99.4	99.0	72.3	82.9	99.1	47.2	92.0
Porosity, %	37.7	38.4	41.9	32.8	35.5	43.3	37.3	45.4
Air filled Poros.,%	27.4	0.2	0.4	9.1	6.1	0.4	19.7	3.6
Water filled Poros.,%	10.4	38.2	41.5	23.7	29.4	42.9	17.6	41.8
Void Ratio	0.61	0.62	0.72	0.49	0.55	0.76	0.59	0.83
Series	1	2	3	4	5	6	7	8

Note: If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

Moisture-Density-Porosity Report

Cooper Testing Labs, Inc. (ASTM D 2937)

Job No: 204-013c **Date:** 08/18/09
Client: SCVWD **By:** RU
Project: Permanente Creek, Mountain View - 10244001

Boring:	TB-6	TB-7	TB-7	TB-7	TB-7	TB-7	TB-10	TB-11
Sample:								
Depth, ft:	41.0	2.0	4.0	5.5	36.0	44.0	44.0	3.0
Visual Description:	Brown Lean Clayey SAND	Brown Clayey SAND w/ Gravel	Brown Clayey SAND	Brown Clayey SAND, trace Gravel	Bronw Clayey SAND (Silty)	Brown Sandy CLAY	Reddish Brown Clayey SAND w/ Gravel	Brown Clayey SAND w/ Gravel
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Total Vol cc	153.0	151.1	152.4	149.9	152.4	154.4	155.2	426.4
Vol Solids,cc	92.4	96.8	97.7	91.2	92.5	88.2	103.9	261.5
Vol Voids,cc	60.6	54.3	54.7	58.6	59.9	66.1	51.3	164.9
Moisture, %	24.1	10.1	10.1	11.7	20.6	27.8	17.8	14.9
Wet Unit wt, pcf	126.4	119.0	119.0	114.7	123.5	123.2	133.0	118.8
Dry Unit wt, pcf	101.9	108.1	108.1	102.7	102.4	96.4	112.9	103.5
Saturation, %	99.1	48.8	48.7	49.1	86.1	100.0	97.2	63.6
Porosity, %	39.6	35.9	35.9	39.1	39.3	42.8	33.0	38.7
Air filled Poros.,%	0.4	18.4	18.4	19.9	5.5	0.0	0.9	14.1
Water filled Poros.,%	39.2	17.5	17.5	19.2	33.8	42.8	32.1	24.6
Void Ratio	0.66	0.56	0.56	0.64	0.65	0.75	0.49	0.63
Series	1	2	3	4	5	6	7	8

Note: If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

COOPER
TESTING LABORATORY

Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D 2937)

Job No: 204-012 **Date:** 07/22/09
Client: SCVWD **By:** RU
Project: Permanente Creek, Mountainview View - 10244001

Boring:	TB-12	TB-12	TB-13					
Sample:								
Depth, ft:	3.0	31.0	6.0					
Visual Description:	Brown Clayey SAND w/ Gravel	Brown Silty SAND w/ Gravel	Brown Lean Clayey SAND w/ Gravel					
Actual G_s								
Assumed G_s	2.70	2.70	2.70					
Total Vol cc	152.1	408.1	437.7					
Vol Solids,cc	102.5	286.9	271.3					
Vol Voids,cc	49.6	121.2	166.4					
Moisture, %	9.5	8.8	11.9					
Wet Unit wt, pcf	124.5	129.1	117.0					
Dry Unit wt, pcf	113.7	118.6	104.6					
Saturation, %	52.8	56.4	52.2					
Porosity, %	32.6	29.7	38.0					
Air filled Poros.,%	15.4	13.0	18.2					
Water filled Poros.,%	17.2	16.7	19.9					
Void Ratio	0.48	0.42	0.61					
Series	1	2	3	4	5	6	7	8

Note: If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

Moisture-Density-Porosity Report

Cooper Testing Labs, Inc. (ASTM D 2937)

Job No: 204-013d   **Date:** 08/18/09

Client: SCVWD   **By:** RU

Project: Permanente Creek, Mountain View - 10244001

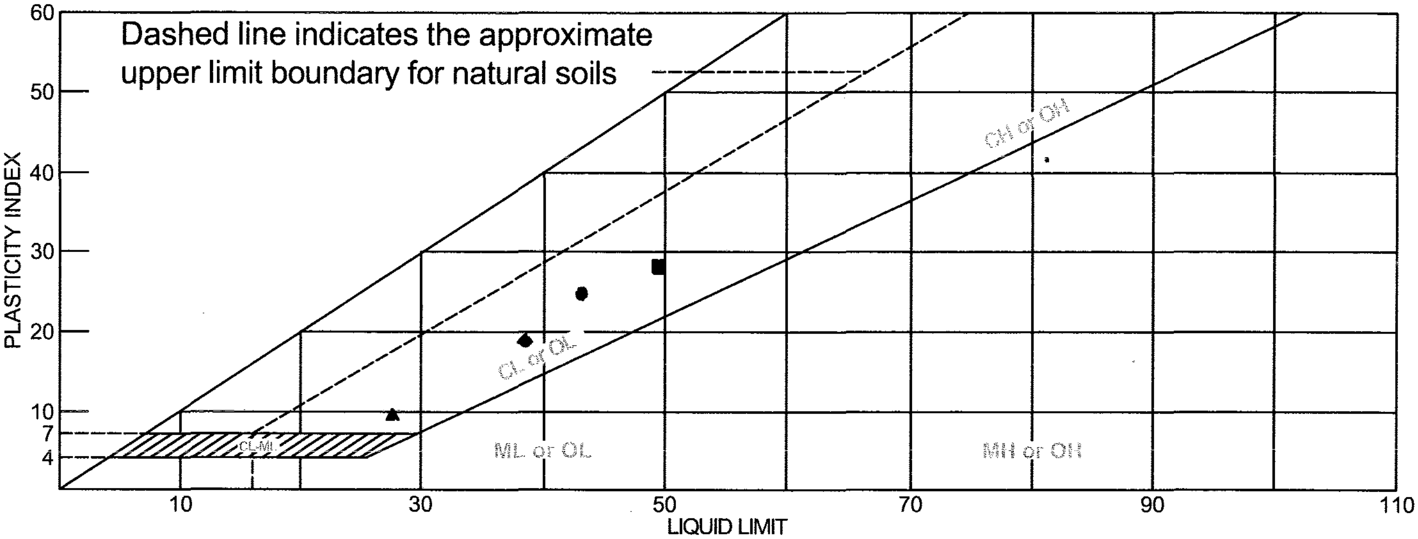
Boring:	TB-11	TB-14	TB-14					
Sample:								
Depth, ft:	10.5	26.0	41.0					
Visual Description:	Brown Silty SAND w/ Gravel (slightly plastic)	Brown Sandy CLAY	Reddish Brown Clayey SAND, trace Gravel					
Actual G_s								
Assumed G_s	2.70	2.70	2.70					
Total Vol cc	153.9	153.6	159.3					
Vol Solids,cc	89.8	96.2	105.5					
Vol Voids,cc	64.1	57.5	53.8					
Moisture, %	11.2	21.9	18.8					
Wet Unit wt, pcf	109.5	128.7	132.7					
Dry Unit wt, pcf	98.4	105.6	111.7					
Saturation, %	42.5	99.0	99.3					
Porosity, %	41.6	37.4	33.8					
Air filled Poros.,%	23.9	0.4	0.2					
Water filled Poros.,%	17.7	37.0	33.5					
Void Ratio	0.71	0.60	0.51					
Series	1	2	3	4	5	6	7	8

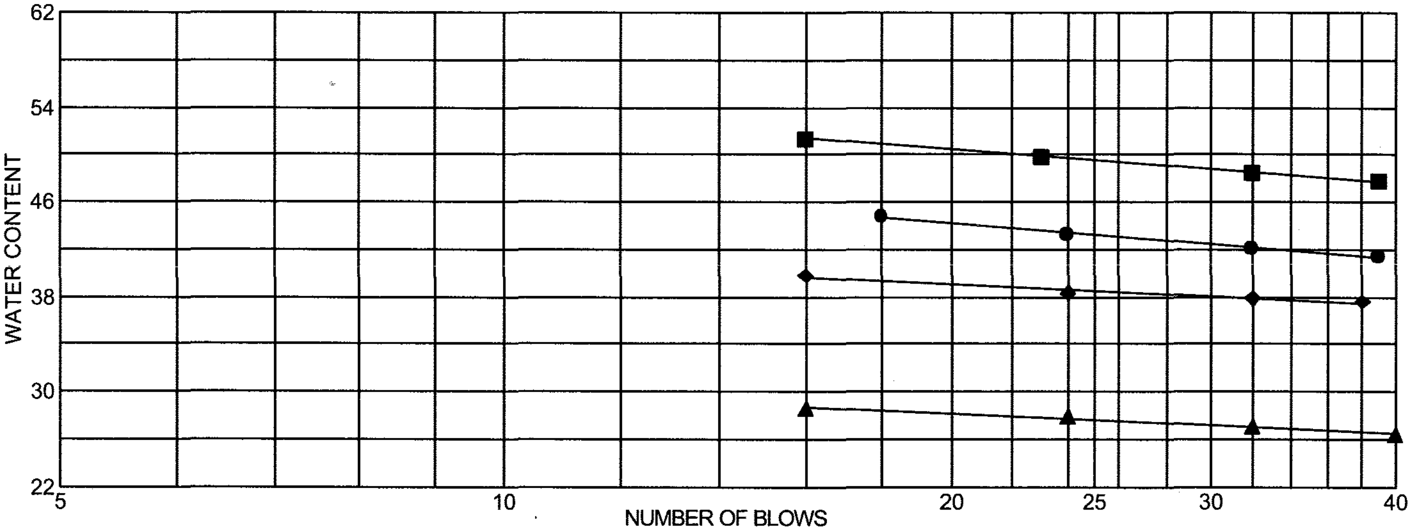
Note: If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

COOPER
TESTING LABORATORY

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Black Sandy Lean CLAY	43.2	18.5	24.7			
■	Brown Sandy Lean CLAY	49.5	21.2	28.3			
▲	Brown Lean Clayey SAND	27.6	18.0	9.6			
◆	Brown Lean Clayey SAND	38.5	19.7	18.8			

Project No. 204-013 **Client:** SCVWD

Project: Permanente Creek, Mountain View - 10244001

● **Source:** TB-1 **Elev./Depth:** 6.0'
■ **Source:** TB-1 **Elev./Depth:** 31.0'
▲ **Source:** TB-2 **Elev./Depth:** 36.0'
◆ **Source:** TB-6 **Elev./Depth:** 41.0'

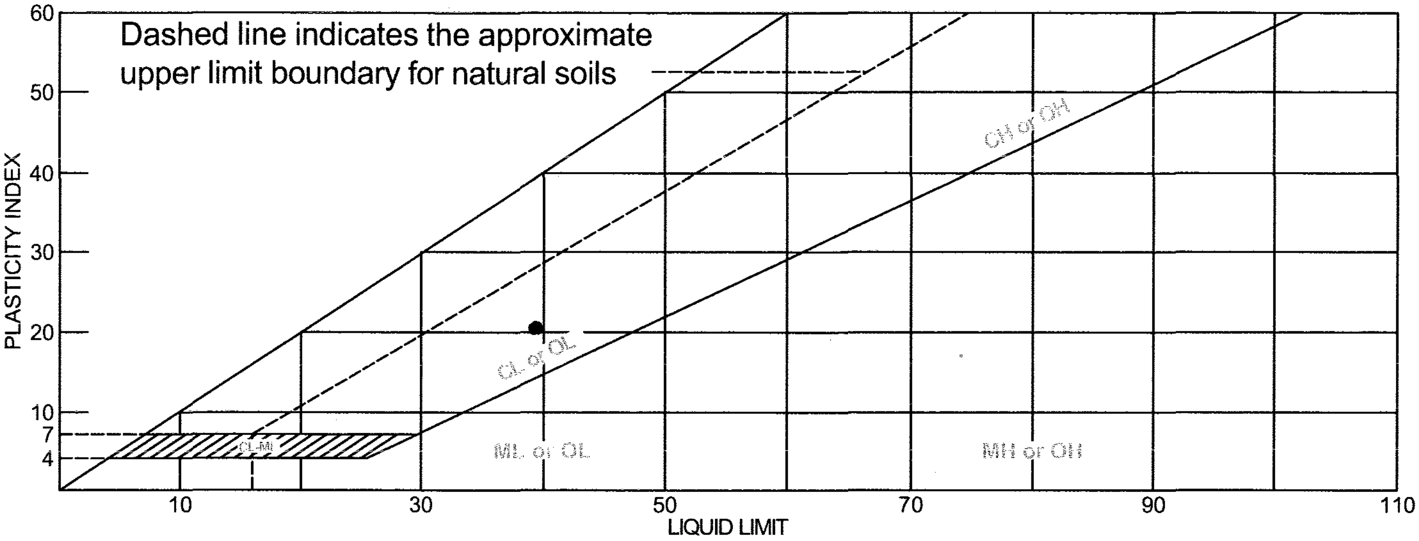
Remarks:
●
■
▲
◆

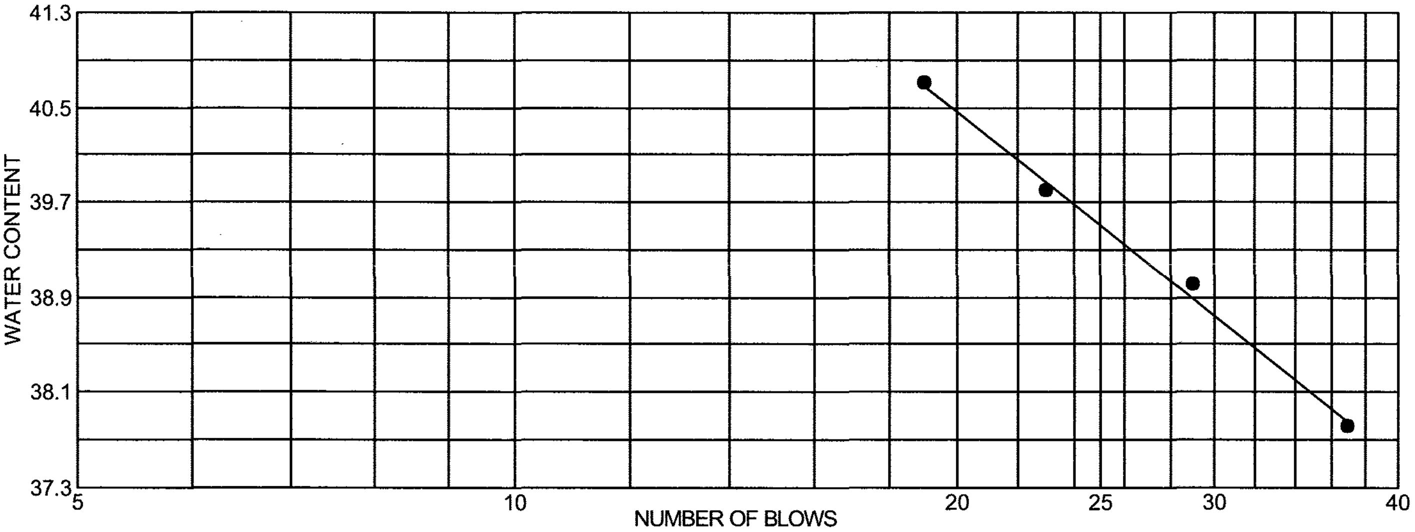
LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown Lean Clayey SAND w/ Gravel	39.5	19.1	20.4			

Project No. 204-012 **Client:** SCVWD

Project: Permanente Creek, Mountain View - 10244001

● **Source:** TB-13 **Elev./Depth:** 6.0'

Remarks:

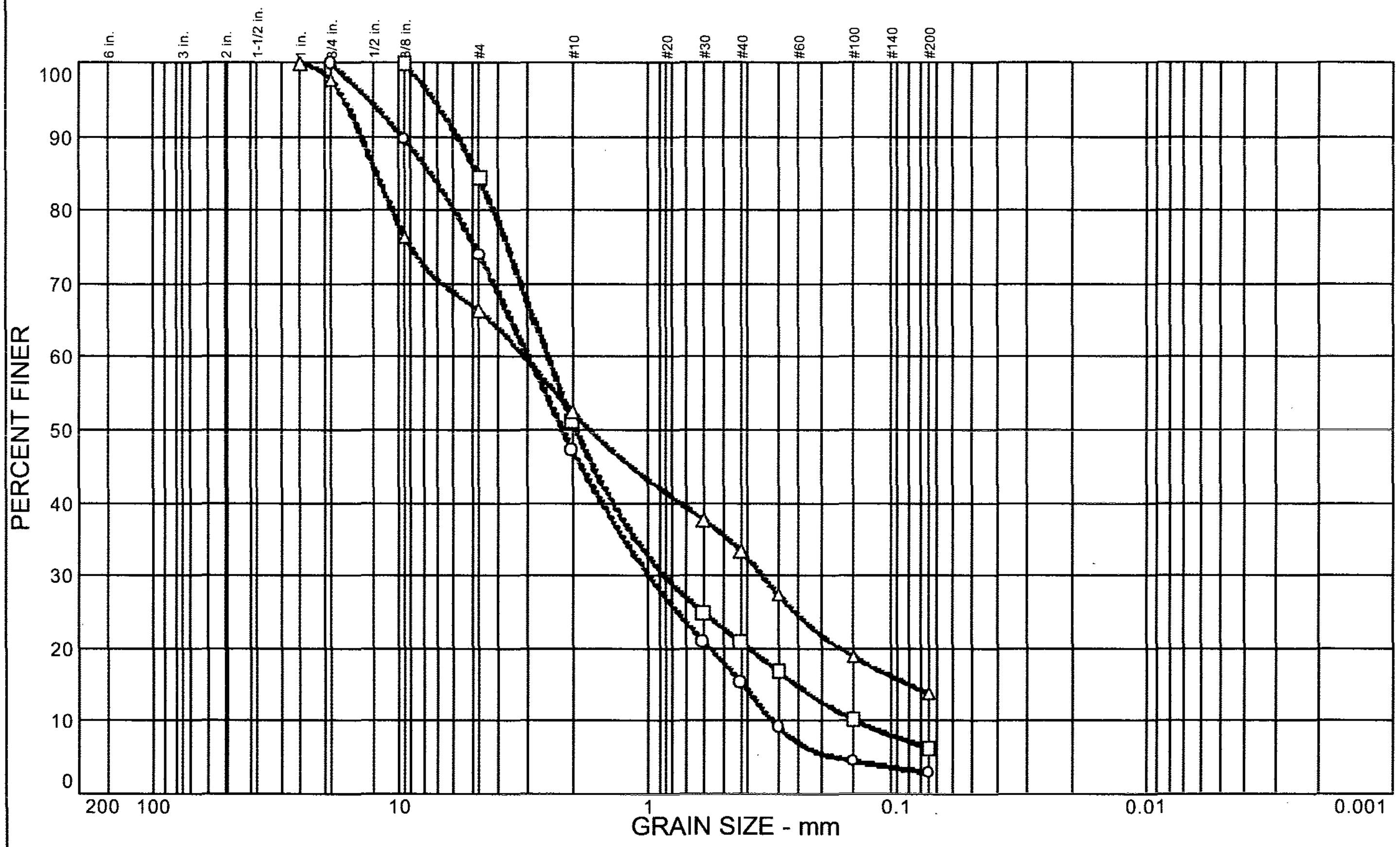
●

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		26.0	71.1	2.9					
□		15.5	78.2	6.3					
△		33.6	52.6	13.8					

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
1"			100.0
3/4"	100.0		97.7
3/8"	89.7	100.0	76.4
	GRAIN SIZE		
D60	3.01	2.52	3.10
D30	0.989	0.857	0.348
D10	0.316	0.146	
	COEFFICIENTS		
C_c	1.03	2.00	
C_u	9.52	17.25	

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
#4	74.0	84.5	66.4
#10	47.3	51.2	52.5
#30	20.9	24.9	37.7
#40	15.3	20.9	33.4
#50	9.2	16.8	27.4
#100	4.5	10.2	18.9
#200	2.9	6.3	13.8

SOIL DESCRIPTION

○ Gray Well-Graded SAND w/ Gravel

□ Gray Well-Graded SAND w/ Silt & Gravel

△ Brown Silty SAND w/ Gravel

REMARKS:

○

□

△

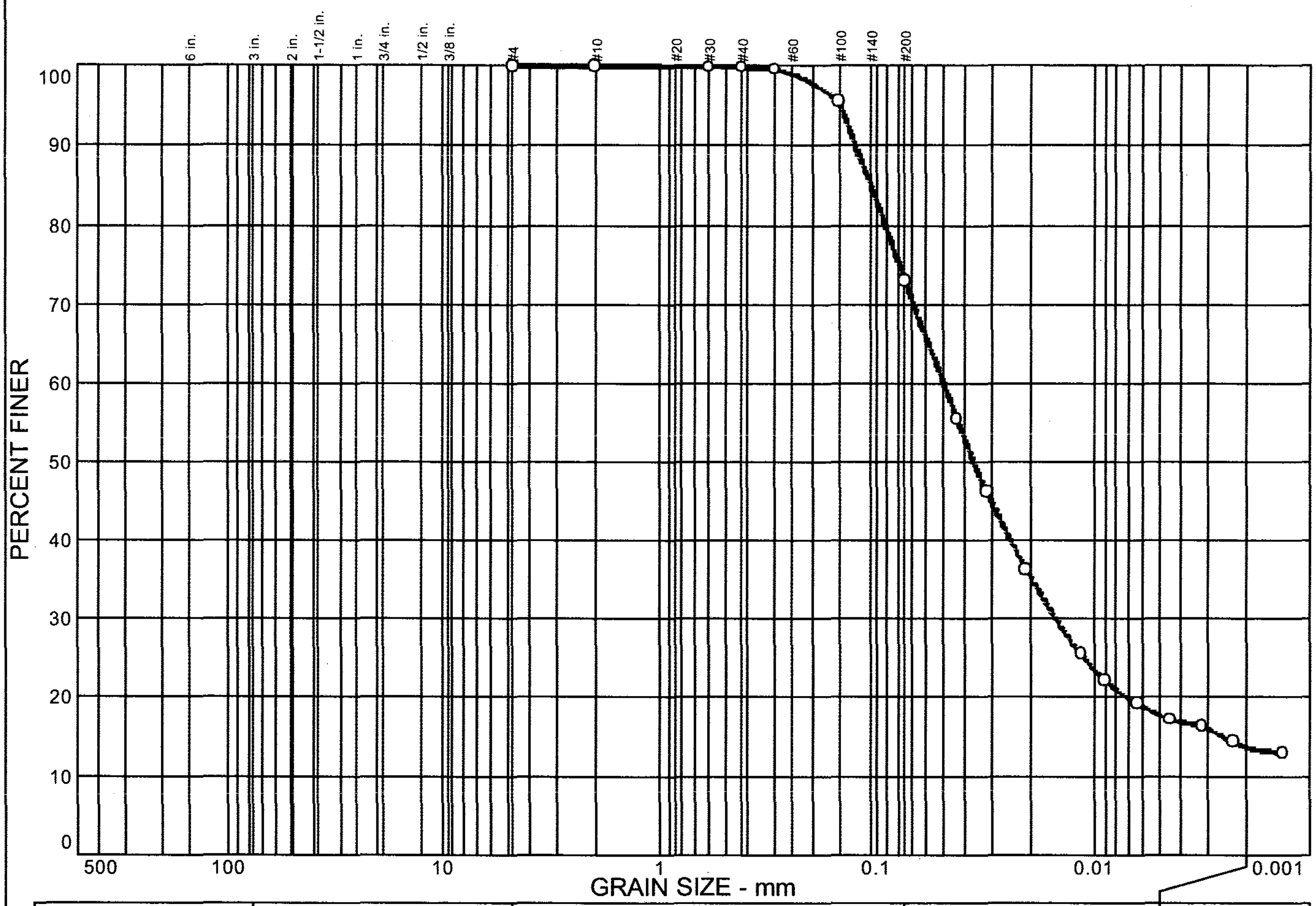
○ Source: TB-1 Elev./Depth: 21-21.5'
□ Source: TB-2 Elev./Depth: 15-16.0'
△ Source: TB-3 Elev./Depth: 6.0'

COOPER TESTING LABORATORY

Client: SCVWD
Project: Permanente Creek, Mountain View - 10244001

Project No.: 204-013 Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	27.0	59.3	13.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#30	99.8		
#40	99.8		
#50	99.6		
#100	95.5		
#200	73.0		
0.0433 mm.	55.4		
0.0317 mm.	46.2		
0.0208 mm.	36.2		
0.0117 mm.	25.5		
0.0090 mm.	22.1		
0.0063 mm.	19.2		
0.0045 mm.	17.2		
0.0032 mm.	16.3		
0.0023 mm.	14.3		
0.0013 mm.	12.9		

* (no specification provided)

Soil Description

Grayish Brown SILT w/ Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.108 D_{60}= 0.0502 D_{50}= 0.0363
D_{30}= 0.0152 D_{15}= 0.0025 D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

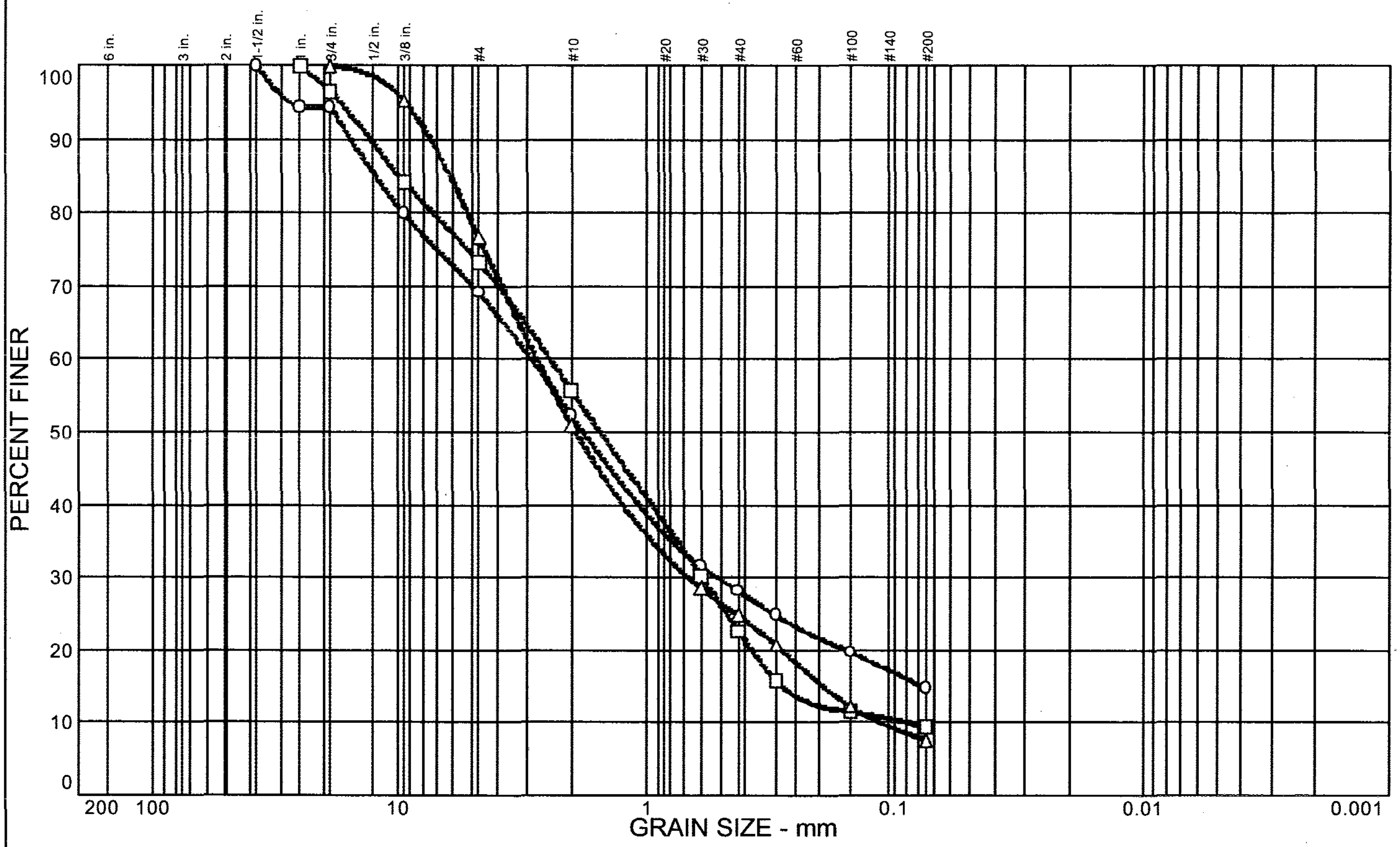
Remarks

Sample No.: **Source of Sample:** TB-2 **Date:**
Location: **Elev./Depth:** 20-22'

COOPER TESTING LABORATORY

Client: SCVWD
Project: Permanente Creek, Mountain View - 10244001
Project No: 204-013 **Figure**

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		30.8	54.5	14.7					
□		26.6	64.0	9.4					
△		23.3	69.1	7.6					

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
1.5"	100.0		
1"	94.4	100.0	
3/4"	94.4	96.5	100.0
3/8"	79.9	84.3	95.3

	GRAIN SIZE ○	GRAIN SIZE □	GRAIN SIZE △
D_{60}	2.90	2.44	2.76
D_{30}	0.517	0.595	0.678
D_{10}		0.0898	0.115

	COEFFICIENTS ○	COEFFICIENTS □	COEFFICIENTS △
C_c		1.62	1.45
C_u		27.16	24.13

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
#4	69.2	73.4	76.7
#10	52.3	55.7	51.0
#30	31.5	30.2	28.5
#40	28.2	22.6	24.8
#50	24.8	15.8	20.7
#100	19.7	11.5	12.2
#200	14.7	9.4	7.6

SOIL DESCRIPTION

○ Brown Silty SAND w/ Gravel

□ Brown Well-Graded SAND w/ Silt & Gravel

△ Brown Well-Graded SAND w/ Silt & Gravel

REMARKS:

○

□

△

○ Source: TB-3	Elev./Depth: 15.5-16.5'
□ Source: TB-4	Elev./Depth: 6.0'
△ Source: TB-4	Elev./Depth: 26.0'

COOPER TESTING LABORATORY

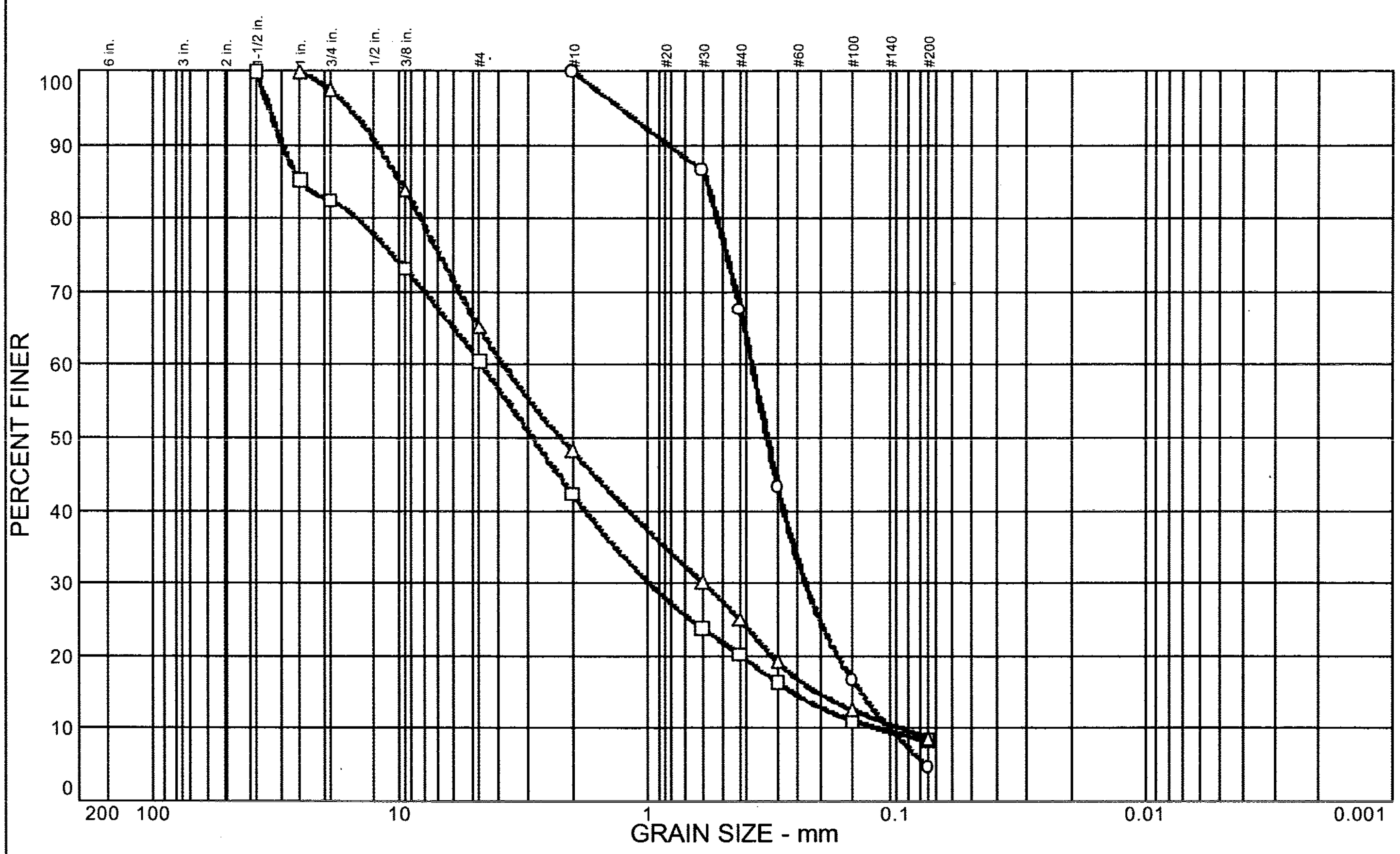
Client: SCVWD

Project: Permanente Creek, Mountain View - 10244001

Project No.: 204-013

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○			95.4	4.6					
□		39.5	52.2	8.3					
△		34.8	56.5	8.7					

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
1.5"		100.0	
1"		85.3	100.0
3/4"		82.5	97.5
3/8"		73.2	83.8
GRAIN SIZE			
D_{60}	0.381	4.63	3.79
D_{30}	0.232	0.978	0.596
D_{10}	0.106	0.118	0.0957
COEFFICIENTS			
C_c	1.34	1.74	0.98
C_u	3.60	39.11	39.54

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
#4		60.5	65.2
#10	100.0	42.3	48.2
#30	86.6	23.8	30.1
#40	67.6	20.2	25.0
#50	43.3	16.3	19.2
#100	16.7	11.1	12.6
#200	4.6	8.3	8.7

SOIL DESCRIPTION

○ Gray Poorly Graded SAND

□ Brown Well-Graded SAND w/ Clay & Gravel

△ Brown Poorly Graded SAND w/ Silt & Gravel

REMARKS:

○

□

△

○ Source: TB-5	Elev./Depth: 2.0'
□ Source: TB-7	Elev./Depth: 16.0'
△ Source: TB-9	Elev./Depth: 5-6.5'

COOPER TESTING LABORATORY

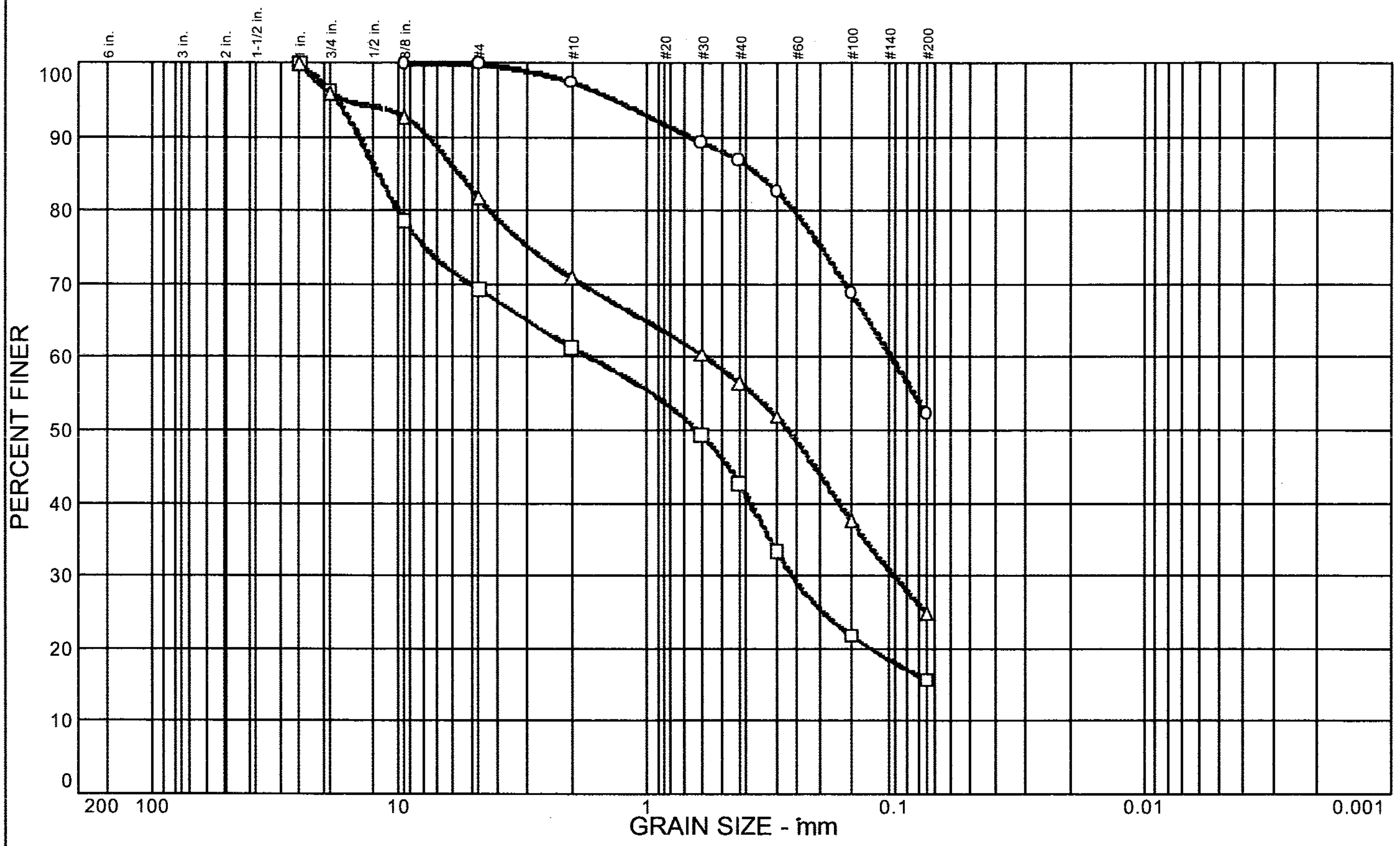
Client: SCVWD

Project: Permanente Creek, Mountain View - 10244001

Project No.: 204-013

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		0.1	47.7	52.2					
□		30.6	53.7	15.7					
△		18.2	57.0	24.8		SM			

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
1"		100.0	100.0
3/4"		96.3	96.0
3/8	100.0	78.7	92.8
	GRAIN SIZE		
D_{60}	0.104	1.68	0.577
D_{30}		0.260	0.101
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
#4	99.9	69.4	81.8
#10	97.4	61.4	70.9
#30	89.3	49.4	60.4
#40	86.8	42.7	56.6
#50	82.7	33.3	51.9
#100	68.7	21.8	37.6
#200	52.2	15.7	24.8

SOIL DESCRIPTION

○ Brown & White Sandy CLAY

□ Brown Silty SAND w/ Gravel

△ Reddish Brown Clayey SAND w/ Gravel

REMARKS:

○

□

△

○ Source: TB-9 Elev./Depth: 15-16.5'

□ Source: TB-9 Elev./Depth: 35-36.5'

△ Source: TB-10 Elev./Depth: 4.0'

COOPER TESTING LABORATORY

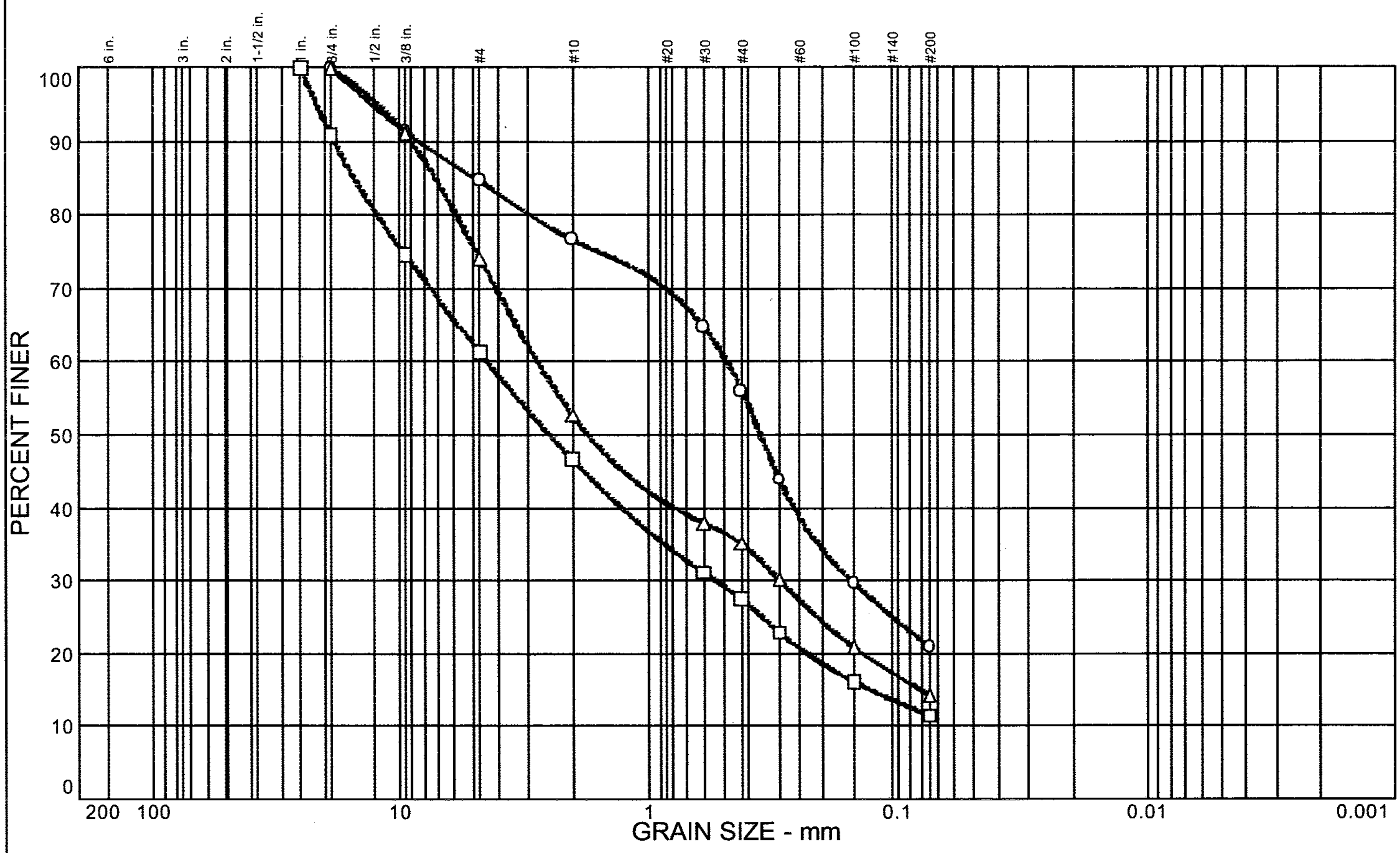
Client: SCVWD

Project: Permanente Creek, Mountain View - 10244001

Project No.: 204-013

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		15.3	63.9	20.8					
□		38.6	50.0	11.4					
△		25.8	59.9	14.3					

SIEVE inches size	PERCENT FINER ○	□	△
1"		100.0	
3/4"	100.0	90.8	100.0
3/8"	91.4	74.7	91.1
	GRAIN SIZE		
D_{60}	0.486	4.40	2.77
D_{30}	0.153	0.537	0.298
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	84.7	61.4	74.2
#10	76.7	46.6	52.7
#30	64.8	31.1	37.9
#40	56.1	27.5	35.1
#50	43.9	22.9	30.1
#100	29.7	16.1	20.8
#200	20.8	11.4	14.3

SOIL DESCRIPTION

○ Brown Silty SAND w/ Gravel

□ Grayish Brown Poorly Graded SAND w/ Silt & Gravel

△ Brown Silty SAND w/ Gravel

REMARKS:

○

□

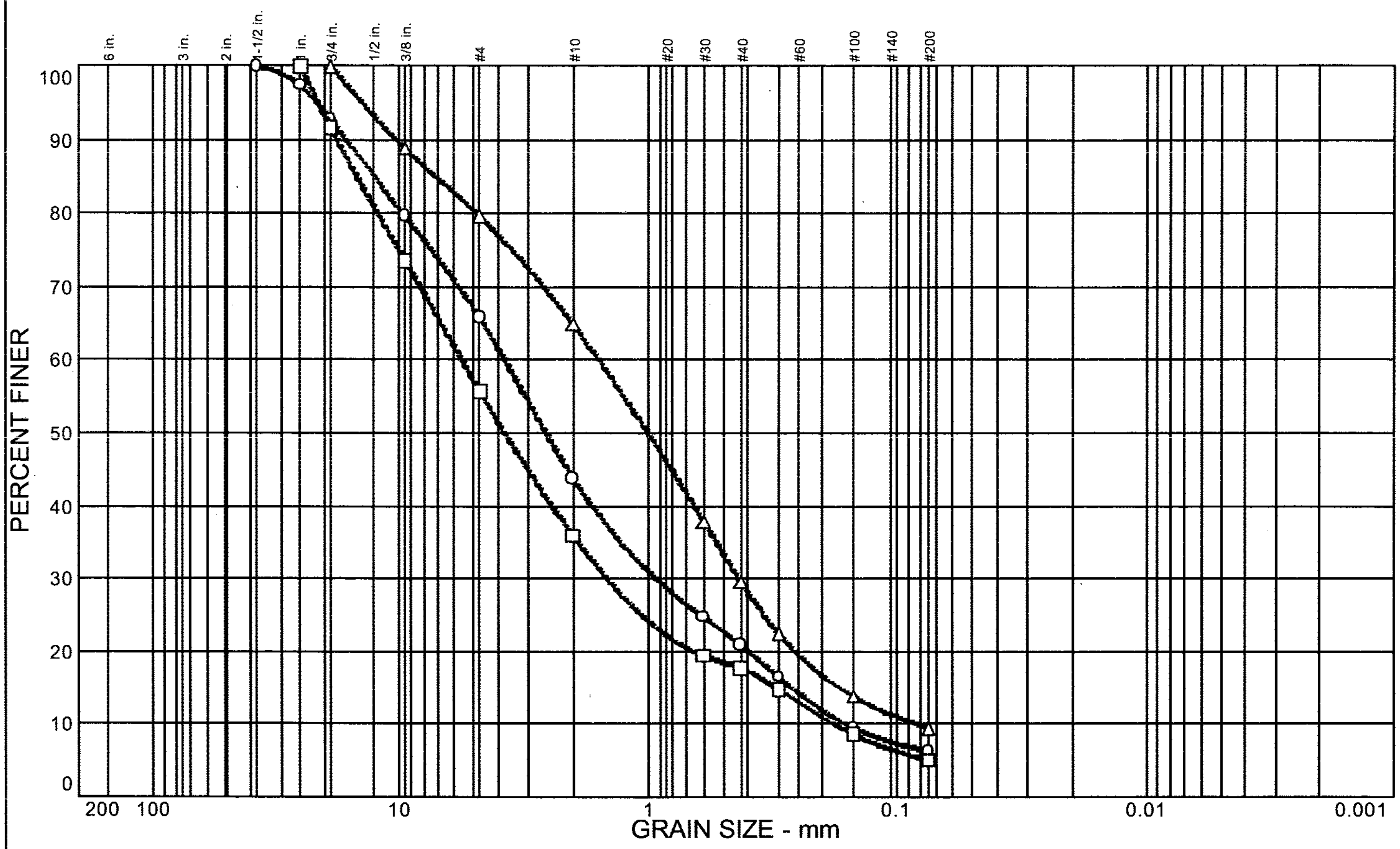
△

○ Source: TB-10 Elev./Depth: 10-11.5'
□ Source: TB-10 Elev./Depth: 25-26.5'
△ Source: TB-11 Elev./Depth: 15-16.5'

COOPER TESTING LABORATORY

Client: SCVWD
Project: Permanente Creek, Mountain View - 10244001
Project No.: 204-013
Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		34.2	59.5	6.3					
□		44.2	50.8	5.0					
△		20.3	70.2	9.5					

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
1.5"	100.0		
1"	97.4	100.0	
3/4"	92.8	91.6	100.0
3/8"	79.7	73.6	88.9
GRAIN SIZE			
D_{60}	3.76	5.61	1.57
D_{30}	0.926	1.46	0.434
D_{10}	0.160	0.179	0.0827
COEFFICIENTS			
C_c	1.42	2.12	1.45
C_u	23.41	31.26	19.03

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
#4	65.8	55.8	79.7
#10	43.8	35.9	64.9
#30	24.7	19.4	37.8
#40	20.9	17.7	29.5
#50	16.5	14.8	22.5
#100	9.5	8.6	13.8
#200	6.3	5.0	9.5

SOIL DESCRIPTION

○ Reddish Brown Well-Graded SAND w/ Clay & Gravel

□ Brown Well-Graded SAND w/ Clay & Gravel

△ Brown Well-Graded SAND w/ Silt & Gravel

REMARKS:

○

□

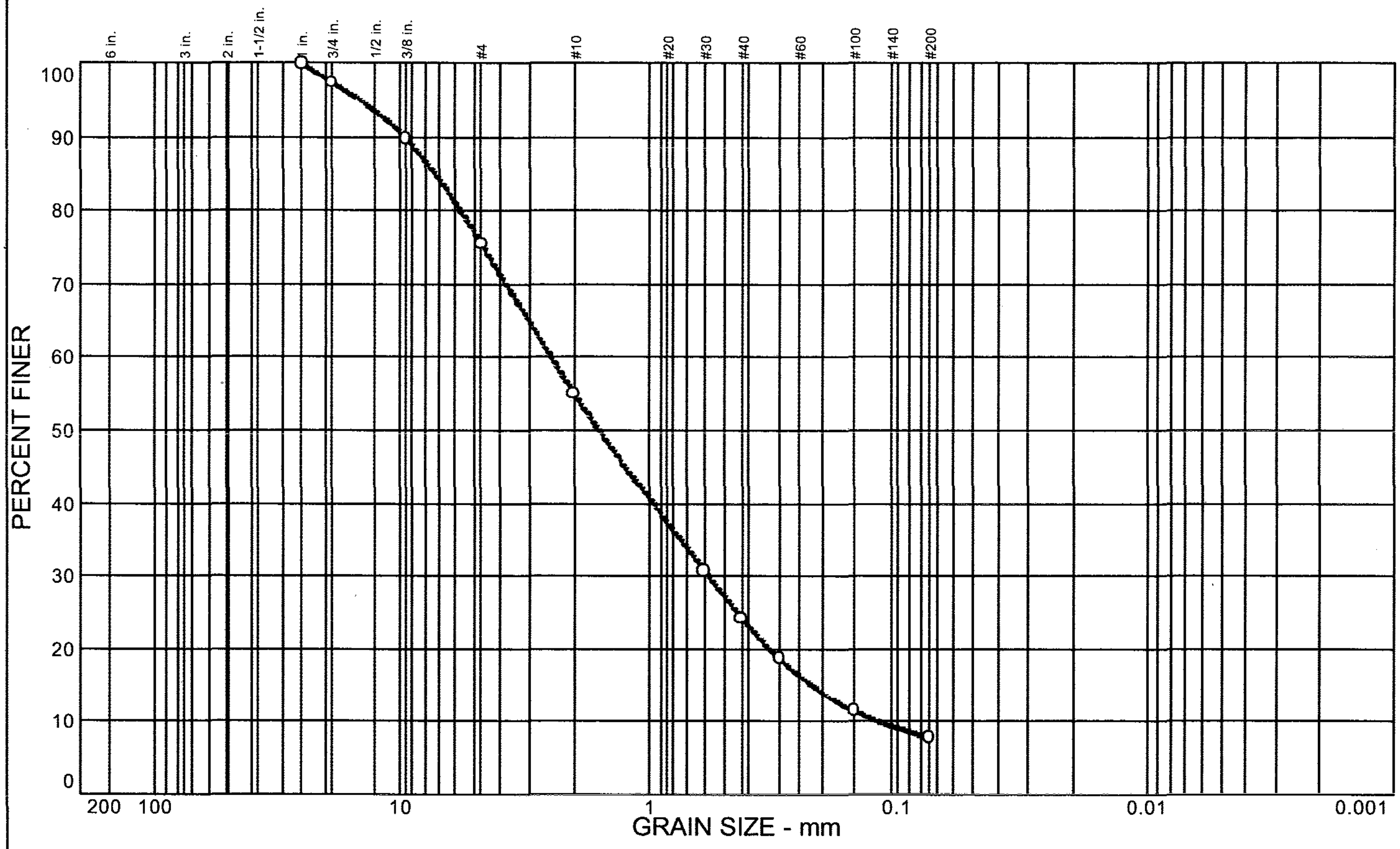
△

○ Source: TB-12 Elev./Depth: 11-11.5'
□ Source: TB-12 Elev./Depth: 15.5'
△ Source: TB-13 Elev./Depth: 20-21.5'

COOPER TESTING LABORATORY

Client: SCVWD
Project: Permanente Creek, Mountain View - 10244001
Project No.: 204-012 Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		24.4	67.8	7.8					

SIEVE inches size	PERCENT FINER ○		
1"	100.0		
3/4"	97.4		
3/8"	89.9		
	GRAIN SIZE		
D_{60}	2.48		
D_{30}	0.576		
D_{10}	0.119		
	COEFFICIENTS		
C_c	1.13		
C_u	20.88		

SIEVE number size	PERCENT FINER ○		
#4	75.6		
#10	55.1		
#30	30.8		
#40	24.2		
#50	18.7		
#100	11.5		
#200	7.8		

SOIL DESCRIPTION

○ Brown Well-Graded SAND w/ Silt & Gravel

REMARKS:

○

○ Source: TB-13

Elev./Depth: 30-31.5'

COOPER TESTING LABORATORY

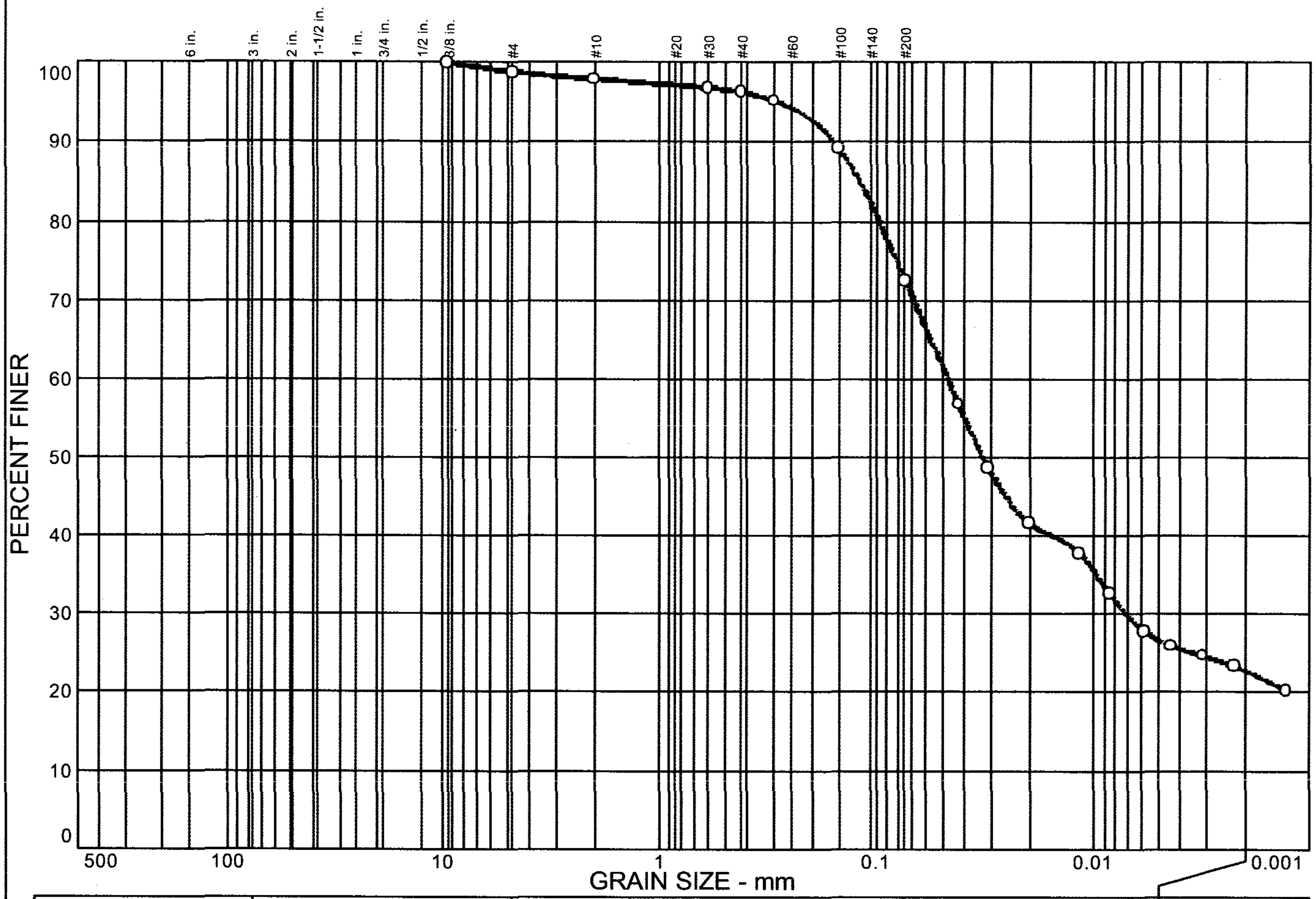
Client: SCVWD

Project: Permanente Creek, Mountain View - 10244001

Project No.: 204-012

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.2	26.3	49.7	22.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	98.8		
#10	97.9		
#30	96.8		
#40	96.3		
#50	95.2		
#100	89.2		
#200	72.5		
0.0424 mm.	56.8		
0.0310 mm.	48.7		
0.0201 mm.	41.6		
0.0118 mm.	37.8		
0.0085 mm.	32.6		
0.0058 mm.	27.8		
0.0044 mm.	25.9		
0.0031 mm.	24.7		
0.0022 mm.	23.3		
0.0013 mm.	20.3		

* (no specification provided)

Soil Description

Brown CLAY w/ Sand

Atterberg Limits

PL=
LL=
PI=

Coefficients

D_{85}= 0.121
D_{60}= 0.0477
D_{50}= 0.0327
D_{30}= 0.0071
D_{15}=
D_{10}=
C_u=
C_c=

Classification

USCS=
AASHTO=

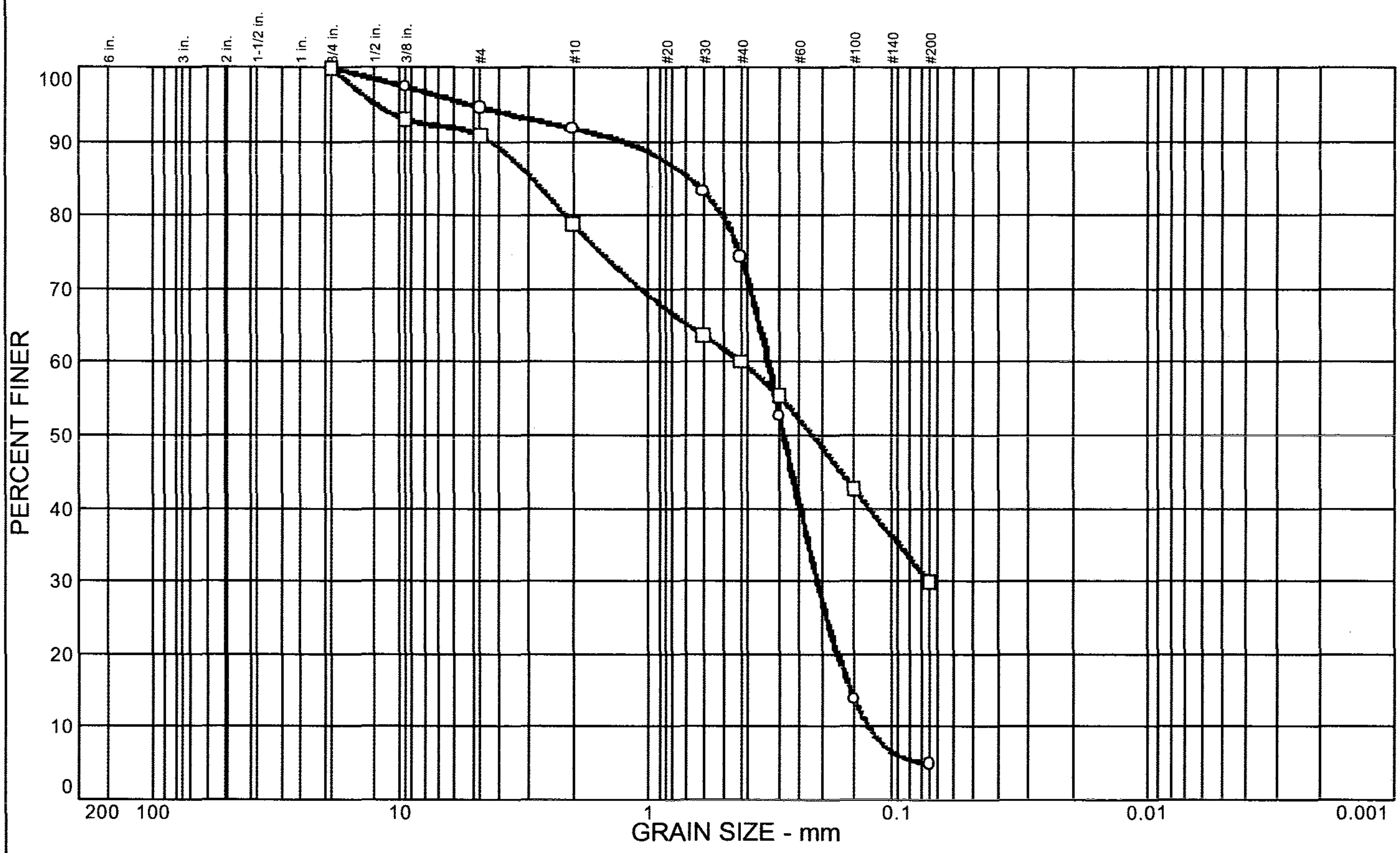
Remarks

Sample No.:
Location:
Source of Sample: TB-14
Date:
Elev./Depth: 5-6.5'

COOPER TESTING LABORATORY

Client: SCVWD
Project: Permanente Creek, Mountain View - 10244001
Project No: 204-013
Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		5.3	89.8	4.9					
□		9.1	60.9	30.0					

SIEVE inches size	PERCENT FINER ○	□	
3/4"	100.0	100.0	
3/8"	97.4	93.0	

	GRAIN SIZE		
D_{60}	0.333	0.421	
D_{30}	0.212	0.0750	
D_{10}	0.131		

	COEFFICIENTS		
C_c	1.03		
C_u	2.55		

SIEVE number size	PERCENT FINER ○	□	
#4	94.7	90.9	
#10	91.8	78.9	
#30	83.3	63.7	
#40	74.3	60.1	
#50	52.7	55.4	
#100	13.8	42.7	
#200	4.9	30.0	

SOIL DESCRIPTION

○ Brown Poorly Graded SAND

□ Brown Clayey SAND

REMARKS:

○

□

○ Source: TB-14 Elev./Depth: 10-11.5'

□ Source: TB-14 Elev./Depth: 30-31.5'

COOPER TESTING LABORATORY

Client: SCVWD

Project: Permanente Creek, Mountain View - 10244001

Project No.: 204-013 Figure

COOPER

#200 Sieve Wash Analysis
ASTM D 1140

Job No.: 204-013a
Project No.: 10244001
Run By: MD
Client: SCVWD
Date: 8/24/2009
Checked By: DC
Project: Permanente Creek, Mountain View

Boring:	TB-1	TB-1	TB-2	TB-3	TB-3	TB-4	TB-4	TB-5
Sample:								
Depth, ft.:	12-13	17-18	5.5	25.5-26.5	40.5-41.5	16-16.5	21-21.5	15.5
Soil Type:	Gray Silty SAND w/ Gravel	Gray SAND w/ Silt and Gravel	Reddish Brown Clayey SAND	Brown Clayey SAND w/ Gravel	Brown Clayey SAND	Brown Clayey SAND	Brown Clayey SAND	Brown Clayey SAND
Wt of Dish & Dry Soil, gm	466.6	613.5	311.1	743.6	463.1	532.6	293.5	540.1
Weight of Dish, gm	164.7	80.3	82.8	313.3	274.3	316.2	79.6	321.9
Weight of Dry Soil, gm	301.9	533.2	228.3	430.3	188.8	216.4	213.9	218.2
Wt. Ret. on #4 Sieve, gm	46.9	129.0	25.1	73.1	13.4	12.1	23.4	21.1
Wt. Ret. on #200 Sieve, gm	245.1	494.2	128.7	329.3	125.6	136.0	163.1	131.4
% Gravel	**15.5**	**24.2**	**11.0**	**17.0**	**7.1**	**5.6**	**10.9**	**9.7**
% Sand	**65.7**	**68.5**	**45.4**	**59.5**	**59.4**	**57.3**	**65.3**	**50.6**
% Silt & Clay	**18.8**	**7.3**	**43.6**	**23.5**	**33.5**	**37.2**	**23.7**	**39.8**

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).

COOPER

#200 Sieve Wash Analysis
ASTM D 1140

Job No.: 204-013b
Project No.: 10244001
Run By: MD
Client: SCVWD
Date: 8/24/2009
Checked By: DC
Project: Permanente Creek, Mountain View

Boring:	TB-6	TB-6	TB-9	TB-9	TB-9	TB-9	TB-11	TB-11
Sample:								
Depth, ft.:	11.0	16	2.5	11-11.5	20.5-21.5	25.5-26.5	3.0	20.5-21.5
Soil Type:	Reddish Brown Sandy CLAY	Brown Clayey SAND	Brown SAND w/ Gravel	Brown SAND w/ Silt and Gravel	Brown Silty SAND w/ Gravel	Grayish Brown Silty SAND w/ Gravel	Brown Clayey SAND w/ Gravel	Brown SAND w/ Silt and Gravel
Wt of Dish & Dry Soil, gm	330.1	547.9	880.9	572.0	445.0	621.8	542.4	389.0
Weight of Dish, gm	83.7	274.2	325.2	324.0	165.9	309.6	311.5	81.9
Weight of Dry Soil, gm	246.4	273.7	555.7	248.0	279.1	312.2	230.9	307.1
Wt. Ret. on #4 Sieve, gm	5.8	20.5	227.6	74.6	85.7	97.3	44.0	108.0
Wt. Ret. on #200 Sieve, gm	99.4	196.5	541.8	223.2	202.9	254.6	152.4	277.4
% Gravel	**2.4**	**7.5**	**41.0**	**30.1**	**30.7**	**31.2**	**19.0**	**35.2**
% Sand	**38.0**	**64.3**	**56.5**	**59.9**	**42.0**	**50.4**	**47.0**	**55.2**
% Silt & Clay	**59.7**	**28.2**	**2.5**	**10.0**	**27.3**	**18.4**	**34.0**	**9.7**

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).

COOPER

#200 Sieve Wash Analysis
ASTM D 1140

Job No.: 204-012
Client: SCVWD
Project: Permanente Creek, Mountain View

Project No.: 10244001
Date: 7/22/2009

Run By: MD
Checked By: DC

Boring:	TB-12	TB-12	TB-12	TB-13	TB-13			
Sample:								
Depth, ft.:	20-21.5	31.0	41.0	15-16.5	40.5			
Soil Type:	Brown CLAY w/ Sand	Brown Silty SAND w/ Gravel	Brown SAND w/ Clay and Gravel	Brown Sandy CLAY	Brown SAND w/ Gravel			
Wt of Dish & Dry Soil, gm	300.8	441.2	634.8	395.2	655.7			
Weight of Dish, gm	156.4	96.5	86.4	81.9	79.9			
Weight of Dry Soil, gm	144.4	344.7	548.4	313.3	575.8			
Wt. Ret. on #4 Sieve, gm	0.3	71.0	139.7	4.1	164.0			
Wt. Ret. on #200 Sieve, gm	28.4	288.4	513.2	110.5	551.7			
% Gravel	**0.2**	**20.6**	**25.5**	**1.3**	**28.5**			
% Sand	**19.5**	**63.1**	**68.1**	**33.9**	**67.3**			
% Silt & Clay	**80.3**	**16.3**	**6.4**	**64.7**	**4.2**			

```
Remarks:  As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is
included is dependent upon both the technician's time available and if there is a significant enough amount of gravel.
The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine
the percentage, especially if there is only a trace amount, (5% or less).
```

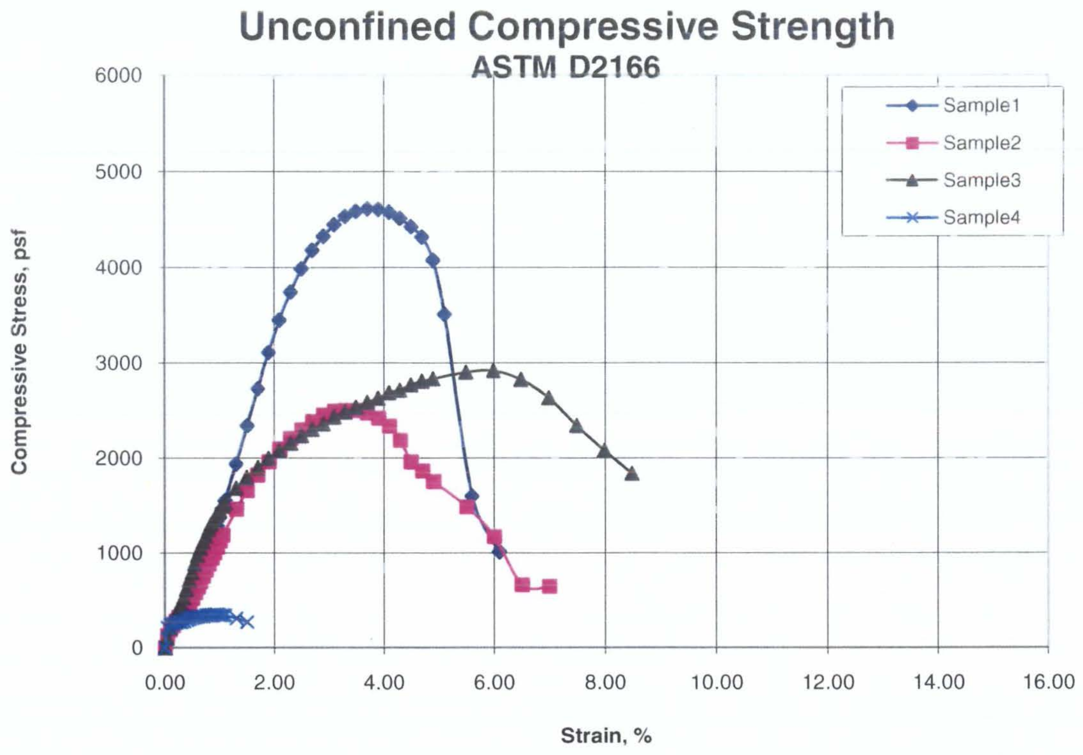
COOPER
TESTING LABORATORY

#200 Sieve Wash Analysis
ASTM D 1140

Job No.: 204-013c
Client: SCVWD
Project: Permanente Creek, Mountain View
Project No.: 10244001
Date: 8/24/2009
Run By: MD
Checked By: DC

Boring:	TB-11	TB-11	TB-14	TB-14				
Sample:								
Depth, ft.:	30.5-31.5	40.5-41.5	15.5-16.5	20.5-21.5				
Soil Type:	Reddish Brown Clayey SAND	Grayish Brown Clayey SAND w/ Gravel	Reddish Brown Sandy SILT	Reddish Brown Clayey SAND				
Wt of Dish & Dry Soil, gm	274.9	355.3	245.9	266.3				
Weight of Dish, gm	84.3	80.4	83.9	83.7				
Weight of Dry Soil, gm	190.6	274.9	162.0	182.6				
Wt. Ret. on #4 Sieve, gm	9.5	71.4	0.0	4.3				
Wt. Ret. on #200 Sieve, gm	106.4	240.6	55.4	101.2				
% Gravel	**5.0**	**26.0**	**0.0**	**2.4**				
% Sand	**50.8**	**61.5**	**34.2**	**53.1**				
% Silt & Clay	**44.2**	**12.5**	**65.8**	**44.6**				

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).



Unconfined Compressive Strength

ASTM D2166

Sample No.:	1	2	3	4	
Unconfined Compressive Strength, psf	4614	2504	2917	350	
Unconfined Compressive Strength, psi	32.0	17.4	20.3	2.4	
Undrained Shear Strength, psf	2307	1252	1458	175	
Failure Strain, %	3.7	3.3	6.0	6.0	
Strain Rate, % per minute	1.0	1.0	1.0	1.0	
Strain Rate, inches/minute	0.05	0.05	0.05	0.05	
Moisture Content, %	20.4	24.6	20.9	7.0	
Dry Density, pcf	108.6	100.3	107.7	119.2	
Saturation, %	99.6	97.6	99.7	45.7	
Void Ratio	0.552	0.681	0.565	0.414	
Specimen Diameter, inches	2.417	2.420	2.390	2.415	
Specimen Height, inches	5.01	5.03	5.00	5.10	
Height to Diameter Ratio	2.1	2.1	2.1	2.1	
Assumed Specific Gravity	2.70	2.70	2.70	2.70	

	Sample Location			
	Boring	Sample	Depth, ft.	Soil Description
1	TB-3		36-36.5	Light Yellowish Brown Sandy CLAY near Cl SAND
2	TB-5		36-36.5	Yellowish Brown Sandy SILT
3	TB-7		26.0	Light Brownish Gray Sandy CLAY/ Clayey SAND
4	TB-10		36.0	Brown SAND w/ Silt & Gravel

Job No.:	204-013	Type of Sample	Undisturbed
Client:	SCVWD		
Project:	10244001	Remarks:	
Date:	8/19/2009 By: MD/RU		

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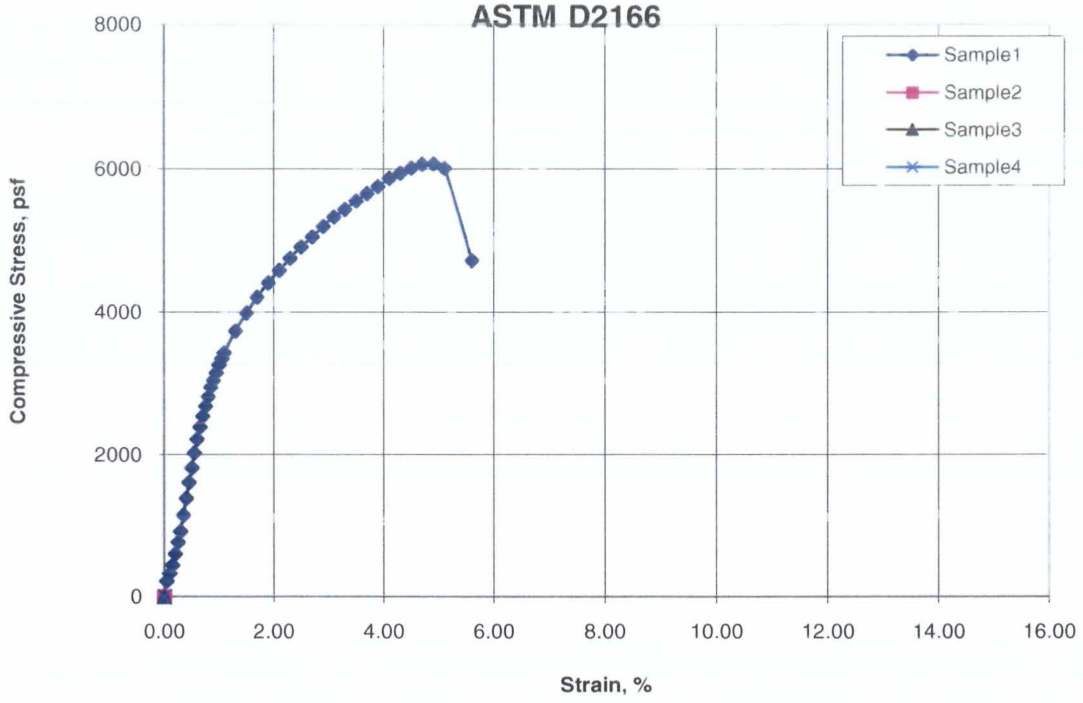
TESTING LABORATORY

Unconfined Compressive Strength

ASTM D2166



Sample No.:	1	2	3	4	
Unconfined Compressive Strength, psf	6074				
Unconfined Compressive Strength, psi	42.2				
Undrained Shear Strength, psf	3037				
Failure Strain, %	4.9				
Strain Rate, % per minute	1.0				
Strain Rate, inches/minute	0.05				
Moisture Content, %	20.5				
Dry Density, pcf	108.1				
Saturation, %	98.8				
Void Ratio	0.559				
Specimen Diameter, inches	2.400				
Specimen Height, inches	4.98				
Height to Diameter Ratio	2.1				
Assumed Specific Gravity	2.70				

	Sample Location			
	Boring	**Sample**	**Depth, ft.**	**Soil Description**
1	TB-12		26.0	Olive CLAY w/ Sand/ Sandy CLAY
2				
3				
4				

Job No.:	204-012	**Type of Sample**	Undisturbed
Client:	SCVWD		
Project:	Permanente Creek - 10244001		
Date:	7/15/2009 **By:** MD/RU		

Remarks:

COOPER
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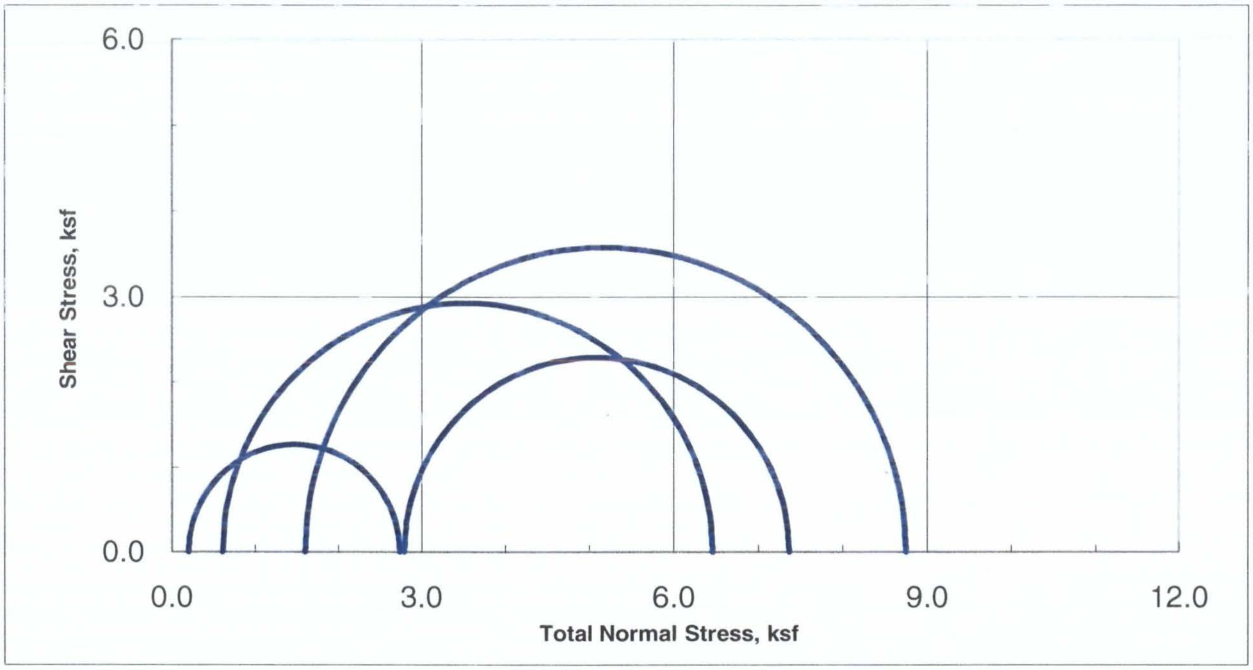
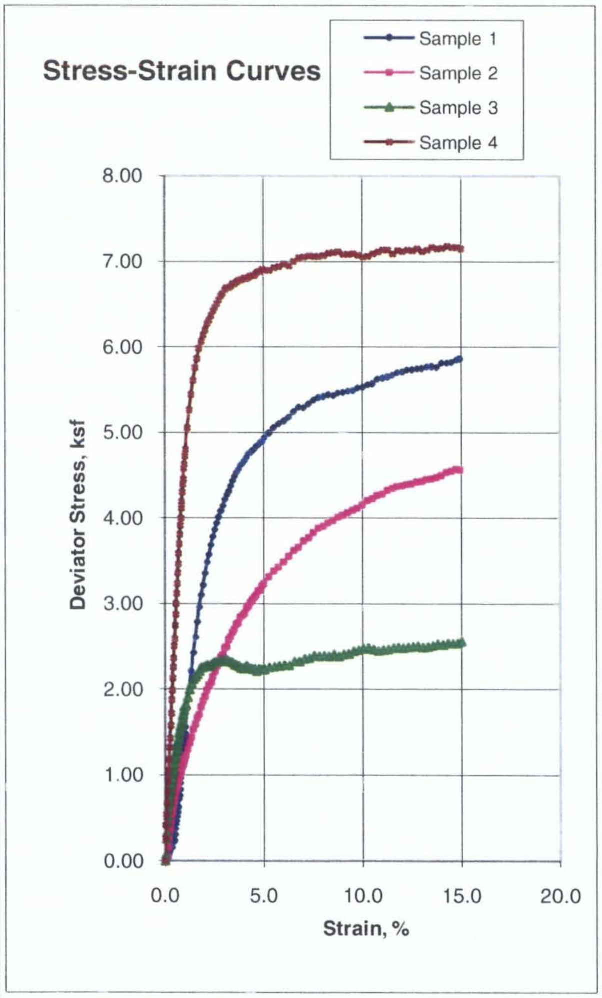
Unconsolidated-Undrained Triaxial Test
ASTM D-2850





Sample Data	1	2	3	4
Moisture %	13.0	20.9	10.6	12.2
Dry Den,pcf	121.0	107.4	89.9	118.3
Void Ratio	0.393	0.569	0.874	0.425
Saturation %	89.4	99.4	32.8	77.7
Height in	5.03	5.06	4.99	5.01
Diameter in	2.42	2.45	2.41	2.42
Cell psi	4.2	19.4	1.4	11.1
Strain %	14.90	14.80	15.00	14.30
Deviator, ksf	5.858	4.574	2.554	7.175
Rate %/min	0.99	1.00	1.00	1.00
in/min	0.050	0.050	0.050	0.050
Job No.:	**204-013a**			
Client:	**SCVWD**			
Project:	**10244001**			
Boring:	TB-2	TB-2	TB-4	TB-5
Sample:				
Depth ft:	6-6.5	31.0	2.0	16

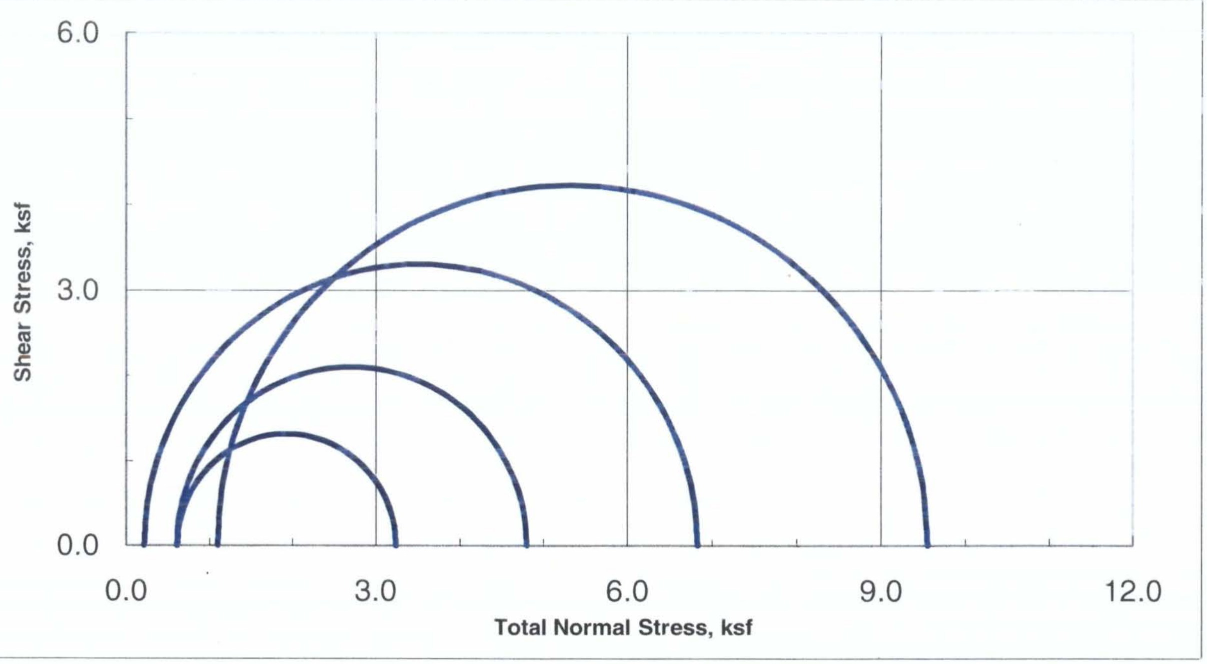
Visual Soil Description

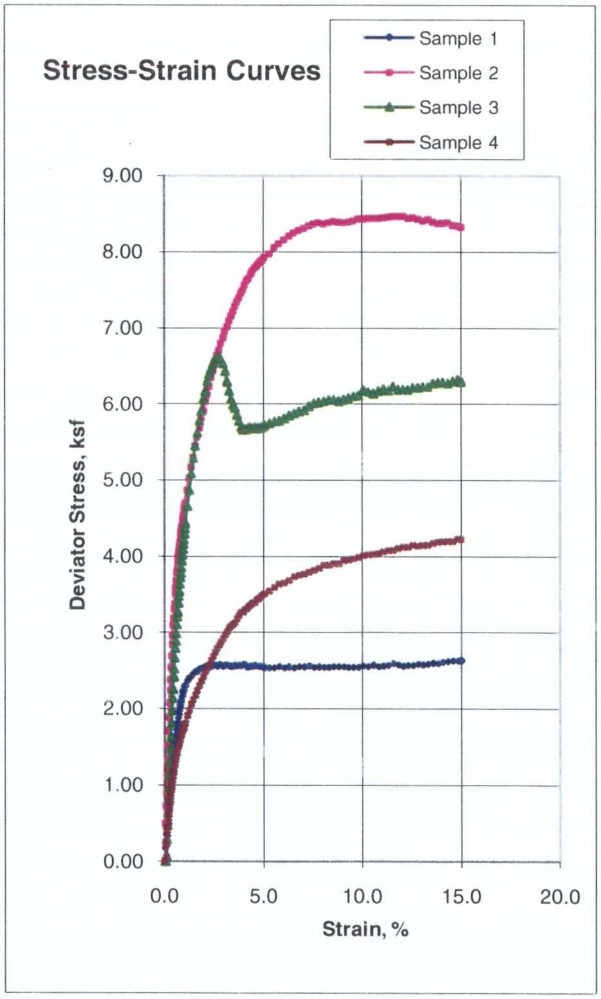
Sample #	
1	Dark Gray Sandy CLAY w/ Gravel/Cl SAND
2	Gray Clayey SAND
3	Brn Sa CLAY, tr roots & root holes (Silty)
4	Br Clayey SAND w/ Gravel

Remarks:

COOPER
TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test
ASTM D-2850





Sample Data				
	1	2	3	4
Moisture %	10.9	13.8	11.6	12.9
Dry Den,pcf	97.7	118.9	113.6	118.0
Void Ratio	0.724	0.418	0.484	0.428
Saturation %	40.5	89.5	64.6	81.4
Height in	5.00	5.00	5.00	4.99
Diameter in	2.42	2.41	2.40	2.40
Cell psi	4.2	7.6	1.5	4.2
Strain %	14.60	11.30	2.70	14.80
Deviator, ksf	2.630	8.473	6.620	4.230
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.050	0.050	0.050
Job No.:	**204-013b**			
Client:	**SCVWD**			
Project:	**10244001**			
Boring:	TB-6	TB-7	TB-10	TB-11
Sample:				
Depth ft:	6.0	11.0	2.0	6.0

Visual Soil Description

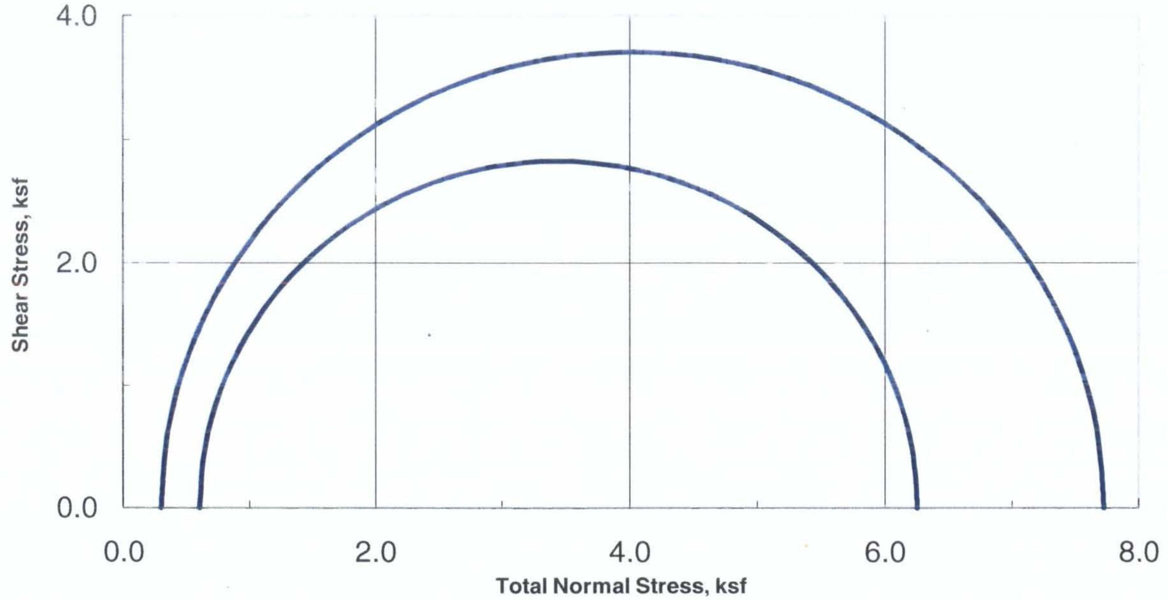
Sample #	
1	Brown Silty SAND (slightly plastic)
2	Brown Clayey SAND w/ Gravel
3	Brown Clayey SAND w/ Gravel
4	Reddish Brown Clayey SAND w/ Gravel

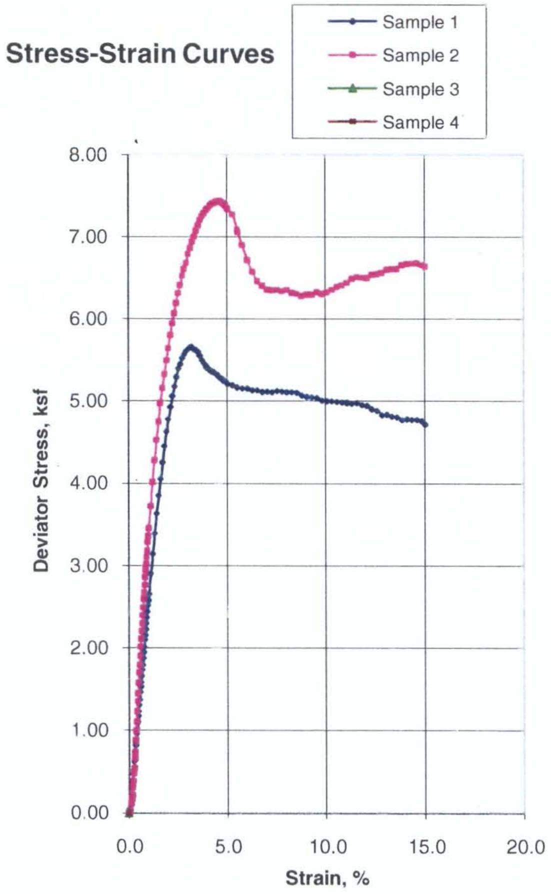
Remarks:

COOPER
TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test

ASTM D-2850





Sample Data	1	2	3	4
Moisture %	11.4	13.3		
Dry Den,pcf	98.2	113.4		
Void Ratio	0.716	0.487		
Saturation %	43.0	73.6		
Height in	5.00	5.00		
Diameter in	2.38	2.38		
Cell psi	4.2	2.1		
Strain %	3.20	4.50		
Deviator, ksf	5.658	7.433		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	204-012			
Client:	SCVWD			
Project:	Permanente Creek - 10244001			
Boring:	TB-12	TB-13		
Sample:				
Depth ft:	6.0-6.5	3.0		

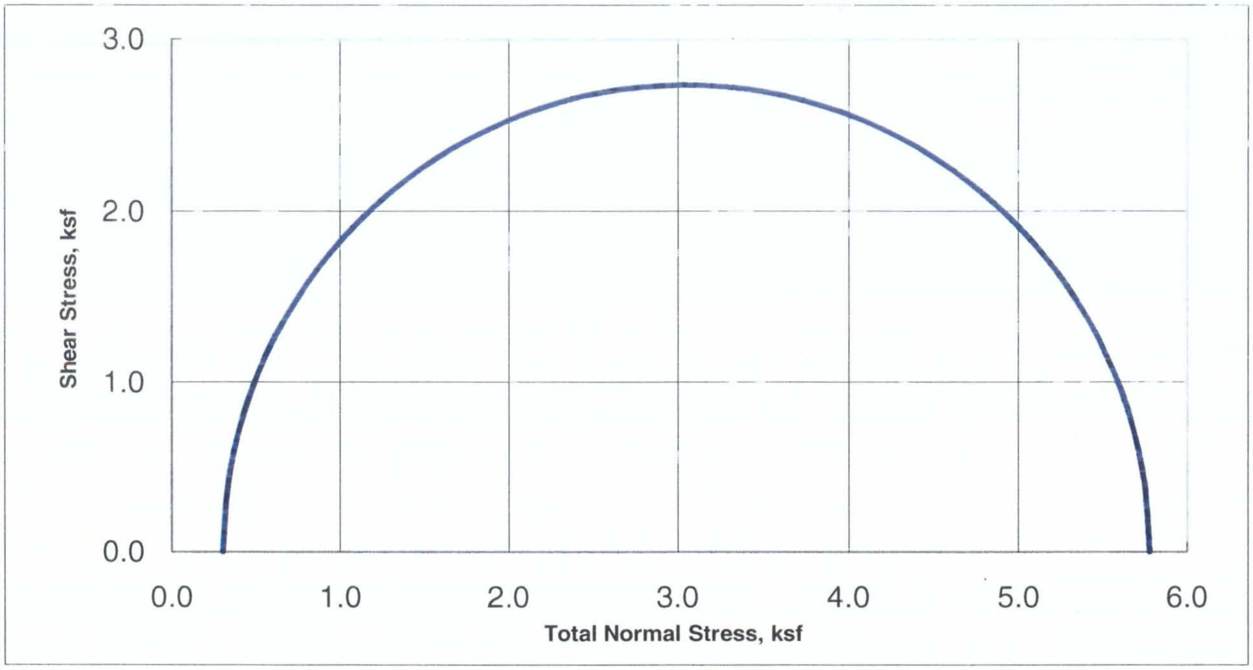
Visual Soil Description

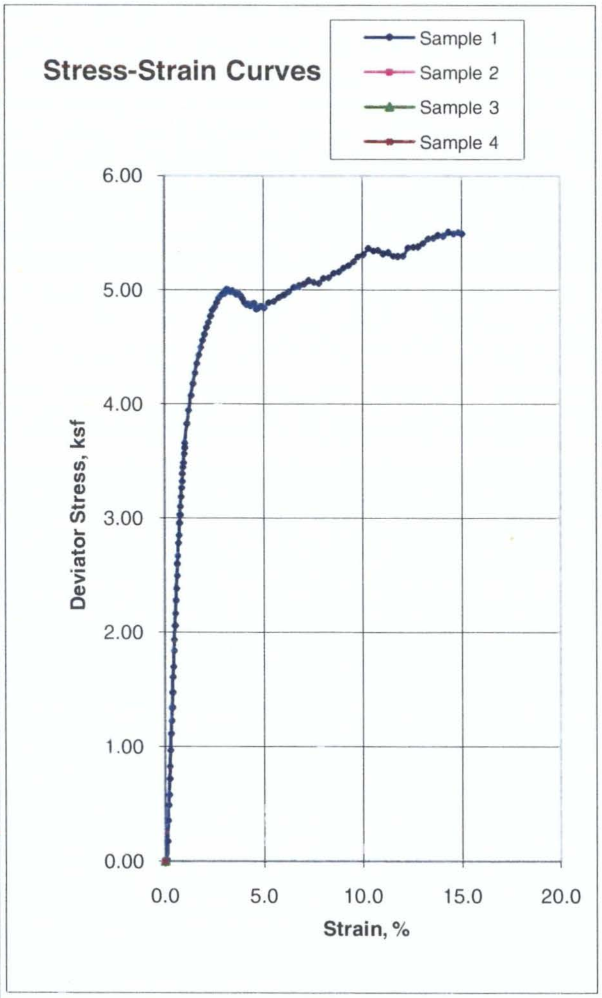
Sample #	
1	Reddish Brown Clayey SAND
2	Dark Reddish Brown Sandy CLAY w/ Gr
3	
4	

Remarks:

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test
ASTM D-2850





Sample Data				
	1	2	3	4
Moisture %	10.6			
Dry Den,pcf	119.5			
Void Ratio	0.410			
Saturation %	69.9			
Height in	5.00			
Diameter in	2.41			
Cell psi	2.1			
Strain %	14.30			
Deviator, ksf	5.508			
Rate %/min	1.00			
in/min	0.050			
Job No.:	204-013c			
Client:	SCVWD			
Project:	10244001			
Boring:	TB-14			
Sample:				
Depth ft:	3.0			

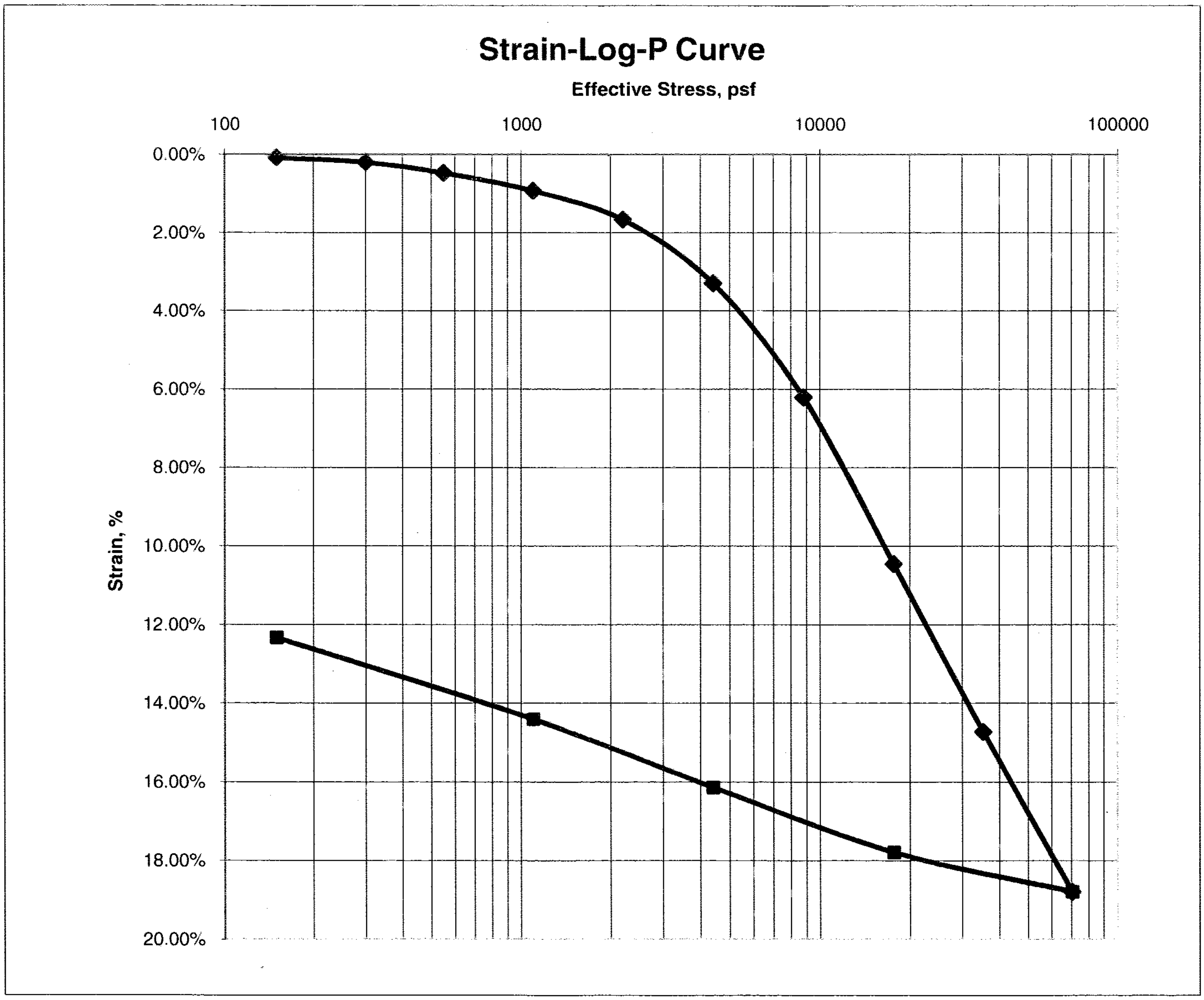
Visual Soil Description	
Sample #	
1	Brown Clayey SAND w/ Gravel
2	
3	
4	

Remarks:

COOPER TESTING LABORATORY

Consolidation Test
ASTM D2435

Job No.:	204-013	**Boring:**	TB-1	**Run By:**	MD
Client:	SCVWD	**Sample:**		**Reduced:**	RU
Project:	10244001	**Depth, ft.:**	35-37	**Checked:**	PJ/DC
Soil Type:	Brown Sandy CLAY, trace Gravel w/ root holes			**Date:**	8/18/2009



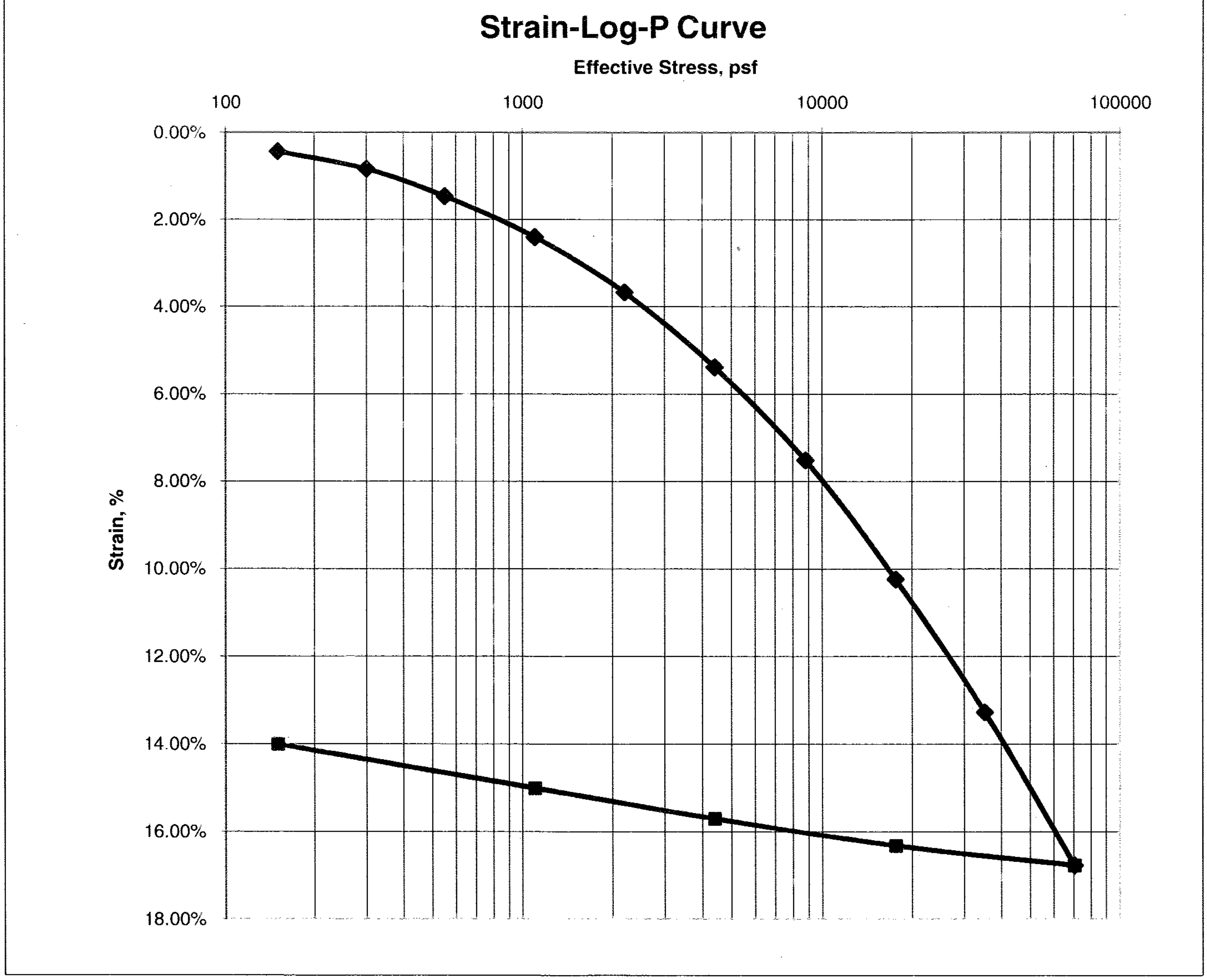
Ass. Gs = 2.7	**Initial**	**Final**
Moisture %:	26.7	21.1
Dry Density, pcf:	95.5	107.5
Void Ratio:	0.764	0.568
% Saturation:	94.5	100

Remarks:

COOPER TESTING LABORATORY

Consolidation Test
ASTM D2435

Job No.:	204-013	**Boring:**	TB-2	**Run By:**	MD
Client:	SCVWD	**Sample:**		**Reduced:**	RU
Project:	10244001	**Depth, ft.:**	20-22	**Checked:**	PJ/DC
Soil Type:	Grayish Brown SILT w/ Sand			**Date:**	8/18/2009

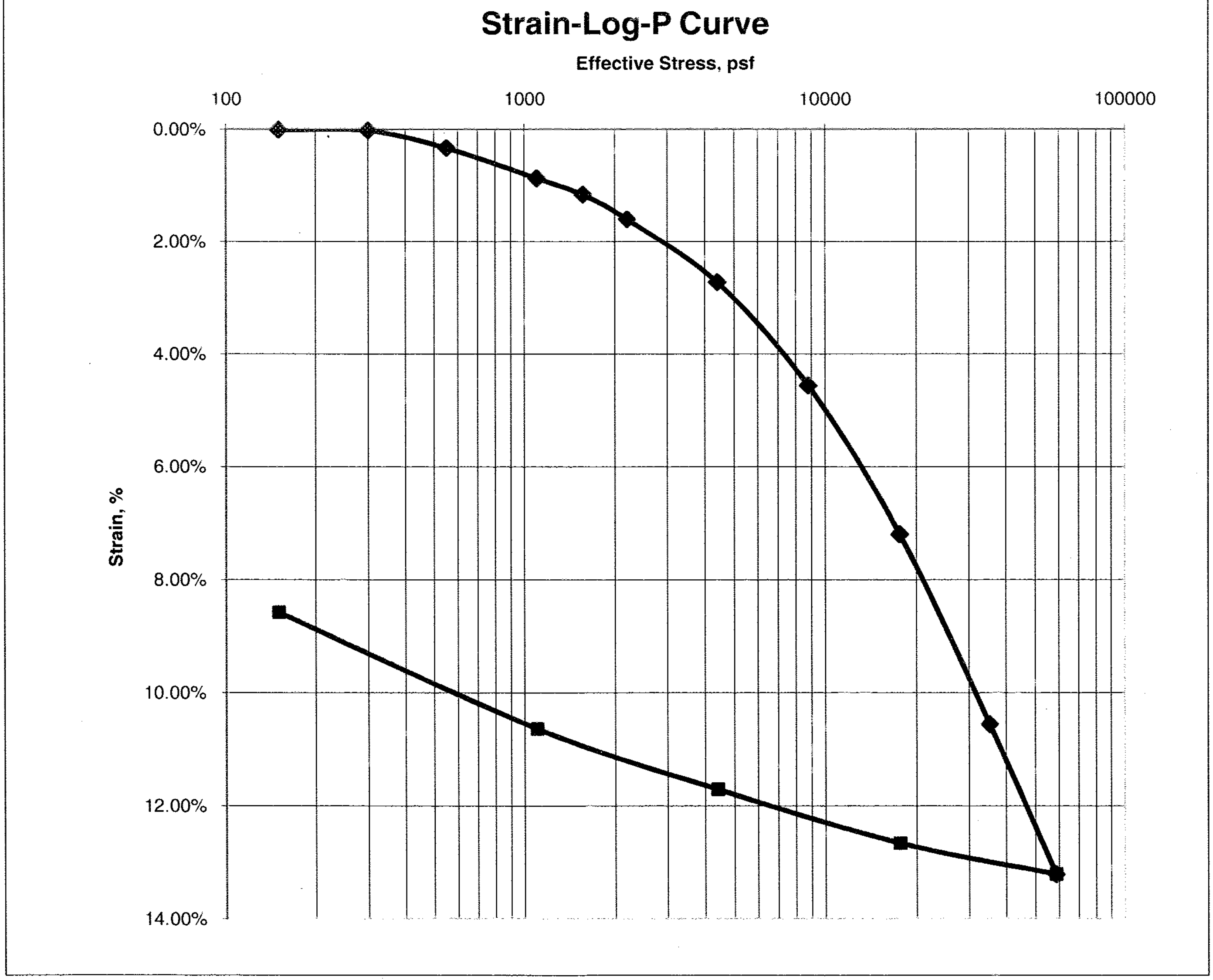


Ass. Gs = 2.7	**Initial**	**Final**
Moisture %:	25.2	18.8
Dry Density, pcf:	98.4	111.9
Void Ratio:	0.712	0.506
% Saturation:	95.4	100

Remarks:

Consolidation Test
ASTM D2435

Job No.:	204-013	**Boring:**	TB-5	**Run By:**	MD
Client:	SCVWD	**Sample:**		**Reduced:**	PJ
Project:	10244001	**Depth, ft.:**	40-42	**Checked:**	PJ/DC
Soil Type:	Brown Silty GRAVEL w/ Sand			**Date:**	8/18/2009



Ass. Gs = 2.7	**Initial**	**Final**
Moisture %:	19.4	17.1
Dry Density, pcf:	110.1	115.5
Void Ratio:	0.531	0.459
% Saturation:	98.8	100

Remarks:

COOPER

Corrosivity Test Summary

CTL # 204-013 **Date:** 8/24/2009 **Tested By:** PJ **Checked:** PJ

Client: SCVWD **Project:** Permanente Creek, Mountain View **Proj. No:** 10244001

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	Qualitative	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	by Lead	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	Acetate Paper	ASTM D2216	
TB-3	-	5.5	-	-	3,020	7	<5	<0.0005	7.0	-	-	13.9	Reddish Brown Clayey SAND
TB-6	-	11.0	-	-	1,670	55	<5	<0.0005	7.3	-	-	10.4	Reddish Brown Sandy CLAY

COOPER
TESTING LABORATORY

Corrosivity Test Summary

CTL # 204-012 **Date:** 7/21/2009 **Tested By:** PJ **Checked:** PJ

Client: SCVWD **Project:** Permanente Creek, Mt. View **Proj. No:** 102440001

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	Qualitative	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	by Lead	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	Acetate Paper	ASTM D2216	
TB-12	-	11-11.5	-	-	6,644	3	<5	<0.0005	6.5	-	-	5.2	Reddish Brown Well-Graded SAND w/ Clay & Gravel

APPENDIX C

Permanente Creek Flood Protection Project

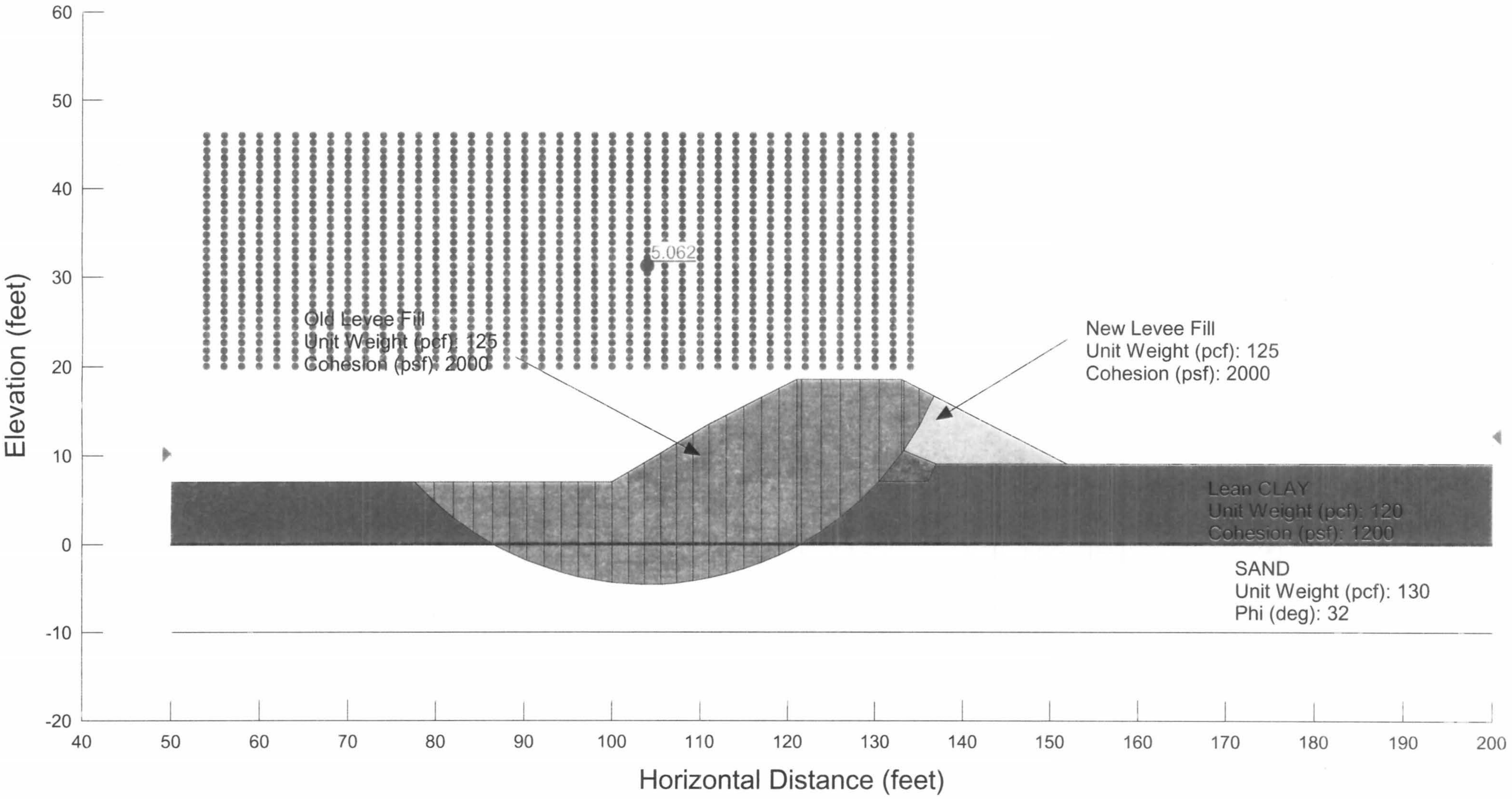
Summary of Groundwater Conditions

C or H-Line Station	Boring	Structure	Piezometer	Drill Date	Groundwater During Drilling		Piezometer Readings After Drilling					
							10/20/2009		1/28/2010		3/12/2010	
					Depth (feet)	Elevation (feet)	Depth (feet)	Elevation (feet)	Depth (feet)	Elevation (feet)	Depth (feet)	Elevation (feet)
C - 107+20	TB-1	None	Yes	6/23/2009	15.9	-0.8	15.7	-0.6	14	1.1	15.2	-0.1
C - 107+55	CPT-1	WB-1 Levee	No	6/23/2009	16.5	0						
C - 110+00	B-4 (Parihk)	EB-2 Floodwall	No	7/29/1999	16.1	0.3						
C - 114+60	B-3 (Parihk)	WB-2 Floodwall	No	7/28/1999	20	-3.9						
C - 116+35	TB-2	WB-2 Floodwall	No	6/24/2009	14	2.9						
C - 122+00	B-2 (Parihk)	EB-3 Floodwall	No	7/28/2009	16.1	0.3						
C - 129+40	B-1 (Parihk)	WB-3 Floodwall	No	7/28/2009	10.8	9.2						
C - 241+45	TB-3	U-frame	No	7/2/2009	42	60.3						
C - 245+30	TB-4	U-frame	No	6/30/2009	39	64.2						
C - 250+35	TB-5	Box culvert	No	7/2/2009	> 45	> 64						
C - 251+00	CPT-6	Box culvert	No	6/24/2009	44.8	63.3						
C - 253+60	TB-6	U-frame	No	7/2/2009	> 45	> 64						
C - 262+00	TB-7	U-frame	Yes	6/25/2009	> 45	> 68	> 45	> 68	> 45	> 68	> 45	> 68
C - 346+00	TB-9	Diversion	Yes	6/29/2009	> 45	> 142	> 45	> 142	> 45	> 142	> 45	> 142
C - 347+20	TB-14	Diversion	No	7/1/2009	> 45	> 146						
H - 3+10	B-4 (SCVWD)	Box culvert	No	12/4/2003	> 45	> 71						
H - 4+00	TB-12	U-frame	No	6/24/2009	> 45	> 72						
H - 12+55	TB-13	Arroyo Floodwall	No	6/30/2009	> 45	> 79						

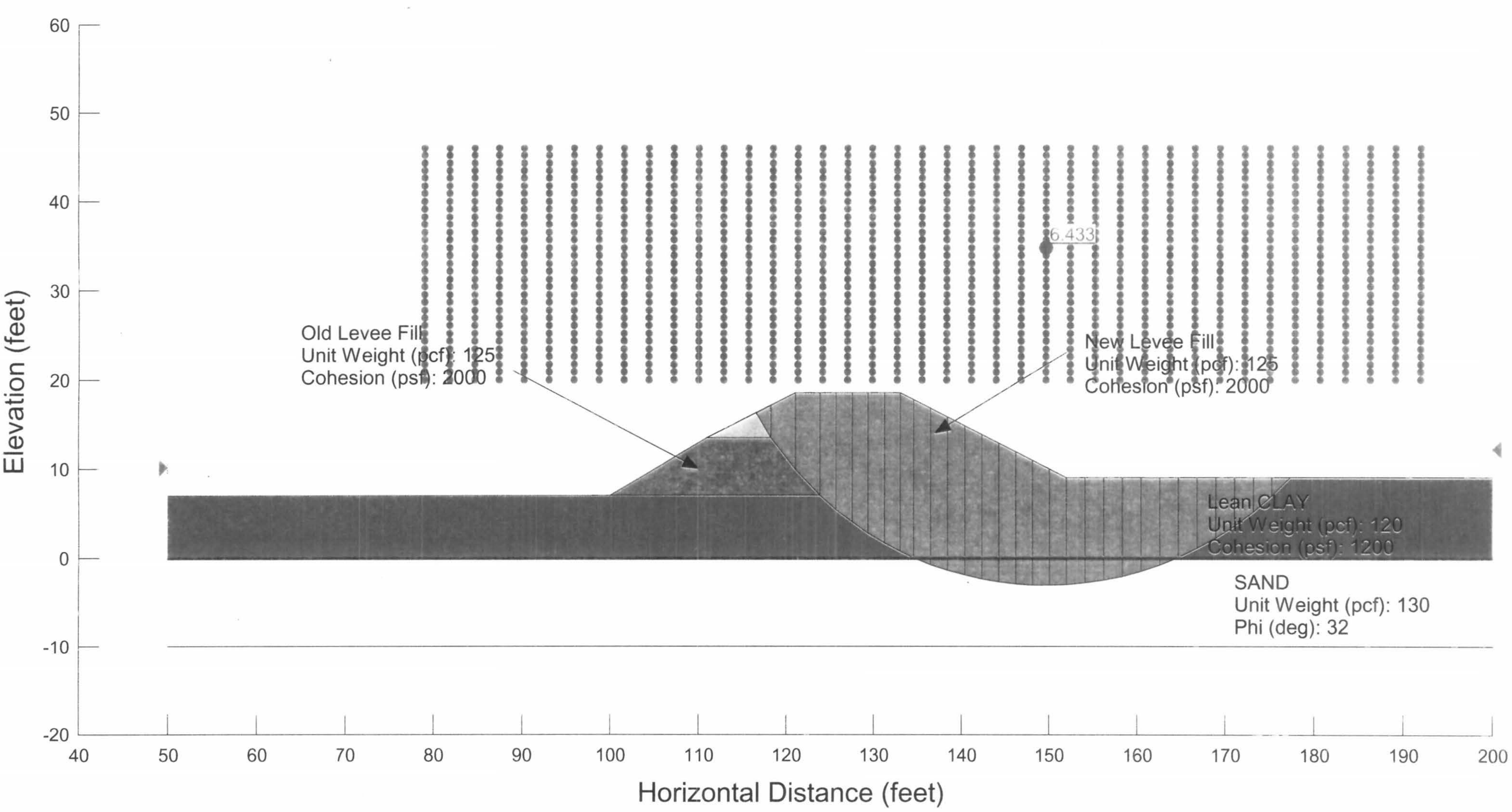
Note: > 45 means that groundwater was not encoutered within the maximum depth of the boring at 45 feet below ground surface

APPENDIX D

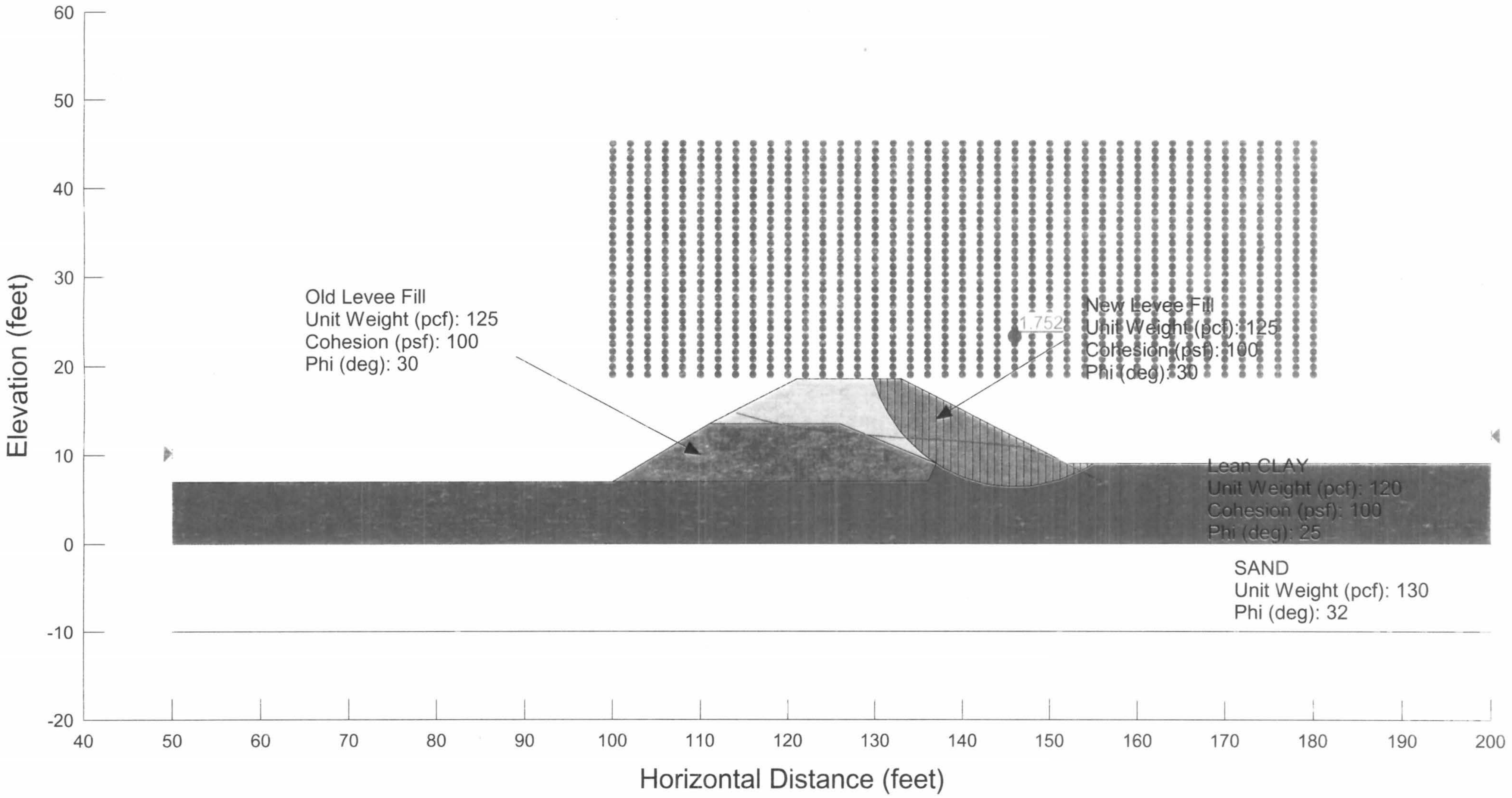
Description: Permanente Creek Flood Protection Project
Comments: Levee WB-1 End of Construction Case (2:1 Slopes)
File Name: WB-1lrA.EOC.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left



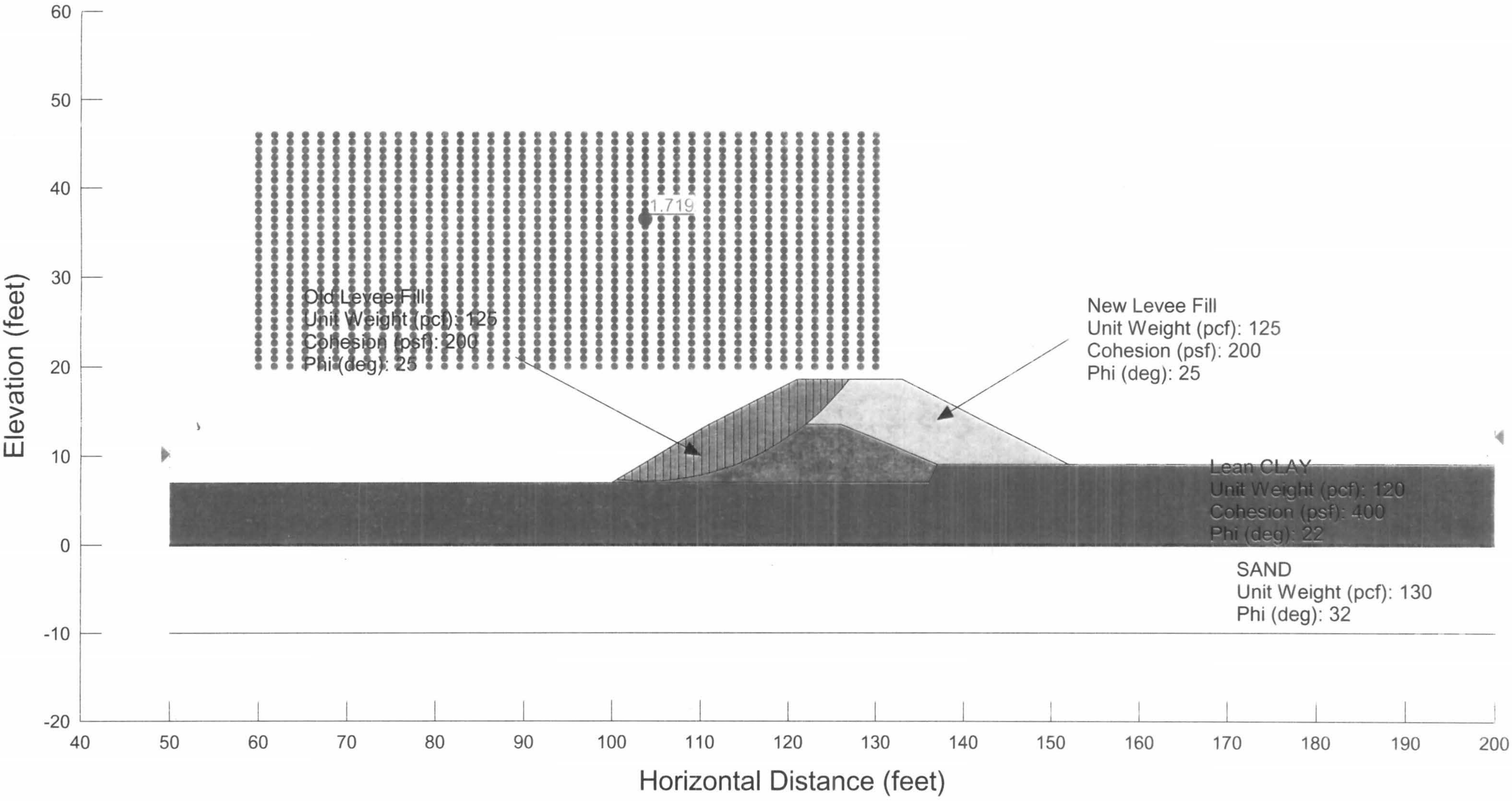
Description: Permanente Creek Flood Protection Project
Comments: Levee WB-1 End of Construction Case (2:1 Slopes)
File Name: WB-1.EOC.slz
Analysis Method: Spencer
Direction of Slip Movement: Left to Right



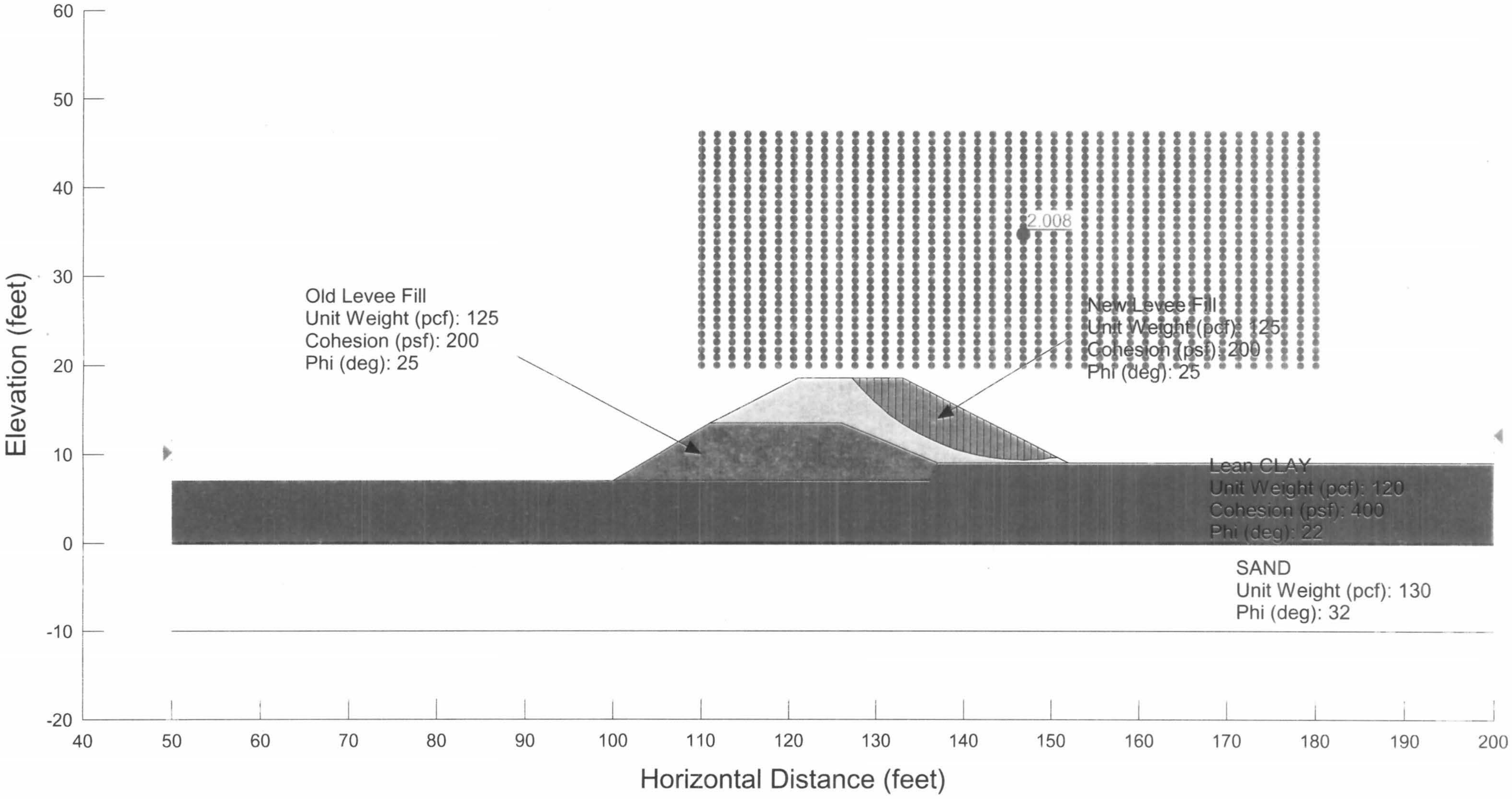
Description: Permanente Creek Flood Protection Project
Comments: Levee WB-1 Steady Seepage Case (2:1 Slopes)
File Name: WB-1lr.SS.slz
Analysis Method: Spencer
Direction of Slip Movement: Left to Right



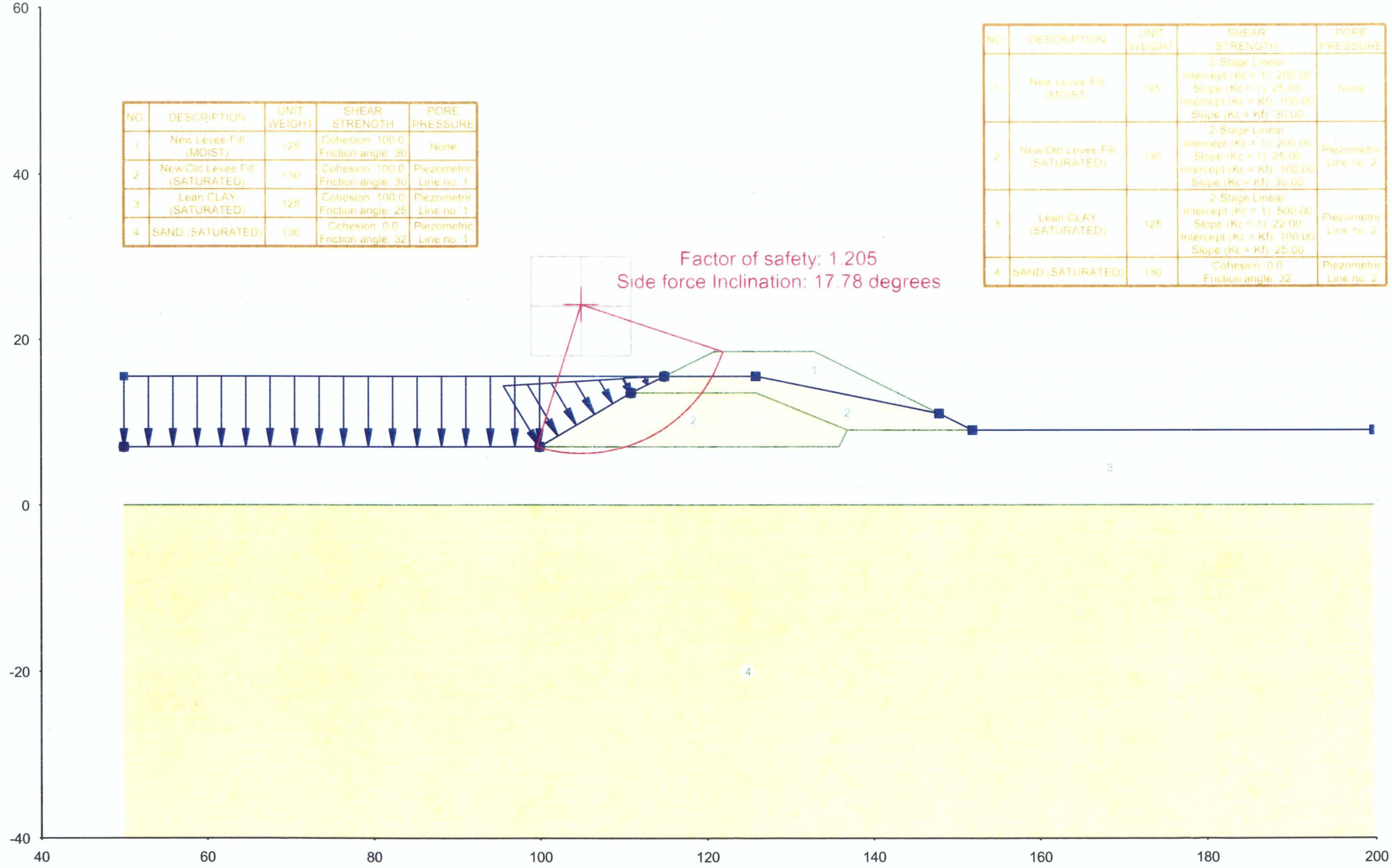
Description: Permanente Creek Flood Protection Project
Comments: Levee WB-1 Pseudo-Static Case, Kh = 0.15 (2:1 Slopes)
File Name: WB-1.PSrl.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left



Description: Permanente Creek Flood Protection Project
Comments: Levee WB-1 Pseudo-Static Case, Kh = 0.15 (2:1 Slopes)
File Name: WB-1.PS.slz
Analysis Method: Spencer
Direction of Slip Movement: Left to Right



Permanente Creek Flood Protection Project, WB-1 Levee with New Fill



Factor of safety: 1.205
Side force Inclination: 17.78 degrees

NO	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	New Levee Fill (MOIST)	125	Cohesion: 100.0 Friction angle: 30	None
2	New/Old Levee Fill (SATURATED)	130	Cohesion: 100.0 Friction angle: 30	Piezometric Line no. 1
3	Lean CLAY (SATURATED)	125	Cohesion: 100.0 Friction angle: 25	Piezometric Line no. 1
4	SAND (SATURATED)	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1

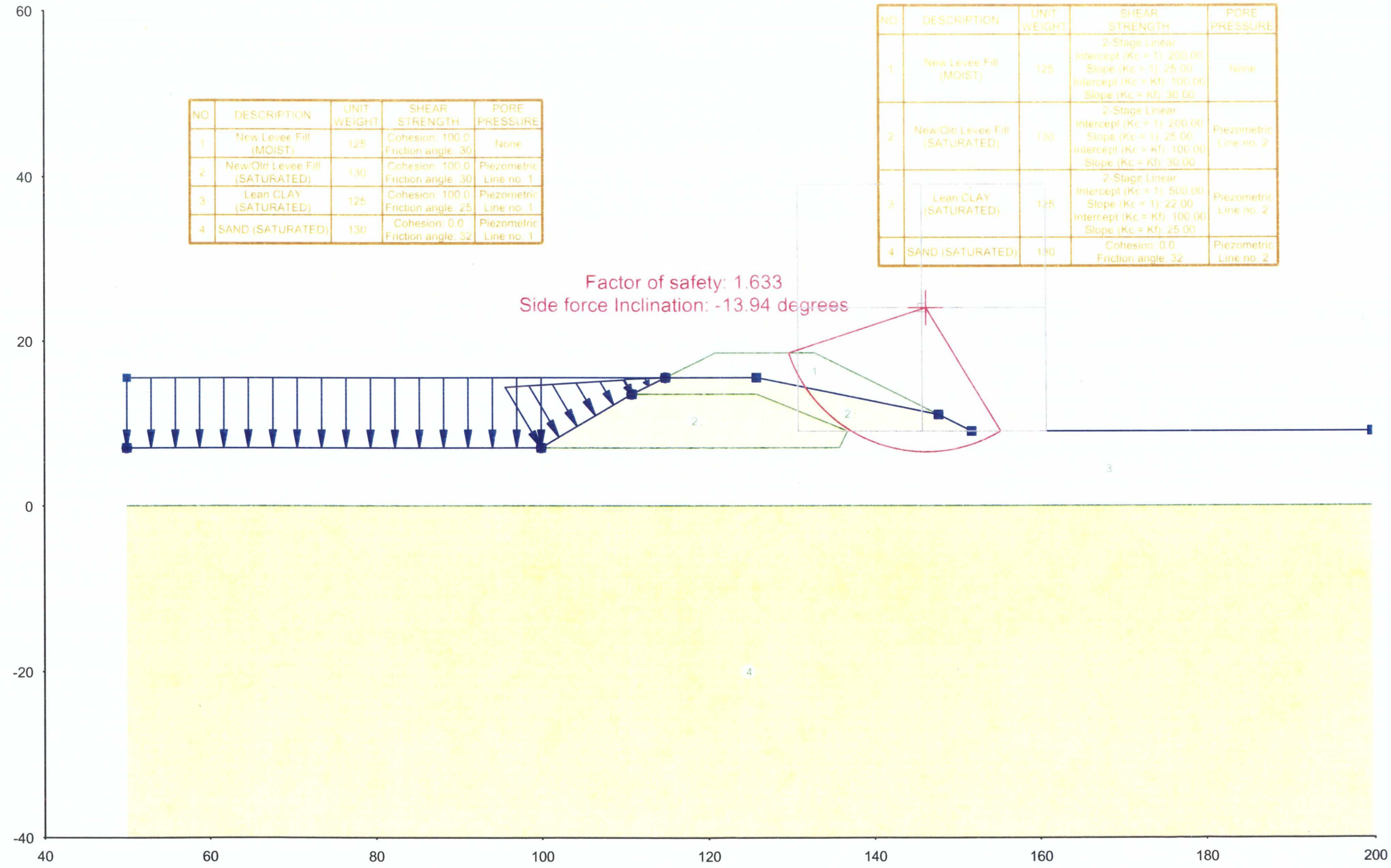
NO	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	New Levee Fill (MOIST)	125	2-Stage Linear Intercept (Kc = 1): 200.00 Slope (Kc = 1): 25.00 Intercept (Kc = Kf): 100.00 Slope (Kc = Kf): 30.00	None
2	New/Old Levee Fill (SATURATED)	130	2-Stage Linear Intercept (Kc = 1): 200.00 Slope (Kc = 1): 25.00 Intercept (Kc = Kf): 100.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 2
3	Lean CLAY (SATURATED)	125	2-Stage Linear Intercept (Kc = 1): 500.00 Slope (Kc = 1): 22.00 Intercept (Kc = Kf): 100.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
4	SAND (SATURATED)	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2

Date: Fri Mar 19 2010

Filename: O:\Geotechnical\Permanente Creek\Slope Stability\WB-1SUDRNa.UT4

Time: 15:20:56

Permanente Creek Flood Protection Project, WB-1 Levee with New Fill



NO	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	New Levee Fill (MOIST)	125	Cohesion: 100.0 Friction angle: 30	None
2	New/Old Levee Fill (SATURATED)	130	Cohesion: 100.0 Friction angle: 30	Piezometric Line no. 1
3	Lean CLAY (SATURATED)	125	Cohesion: 100.0 Friction angle: 25	Piezometric Line no. 1
4	SAND (SATURATED)	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1

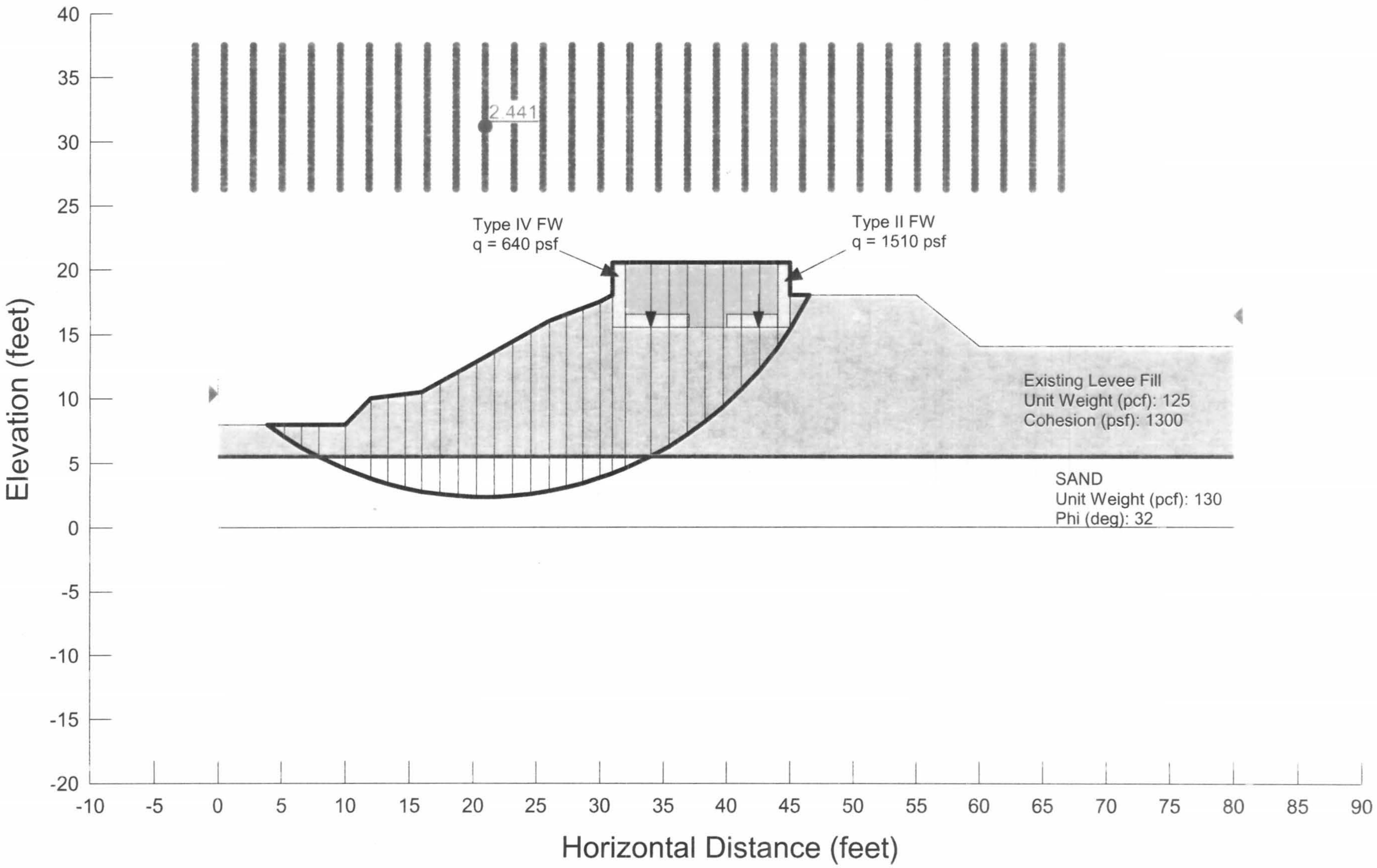
NO	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	New Levee Fill (MOIST)	125	2-Stage Linear Intercept (Kc = 1): 200.00 Slope (Kc = 1): 25.00 Intercept (Kc = Kf): 100.00 Slope (Kc = Kf): 30.00	None
2	New/Old Levee Fill (SATURATED)	130	2-Stage Linear Intercept (Kc = 1): 200.00 Slope (Kc = 1): 25.00 Intercept (Kc = Kf): 100.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 2
3	Lean CLAY (SATURATED)	125	2-Stage Linear Intercept (Kc = 1): 500.00 Slope (Kc = 1): 22.00 Intercept (Kc = Kf): 100.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
4	SAND (SATURATED)	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2

Date: Fri Mar 19 2010

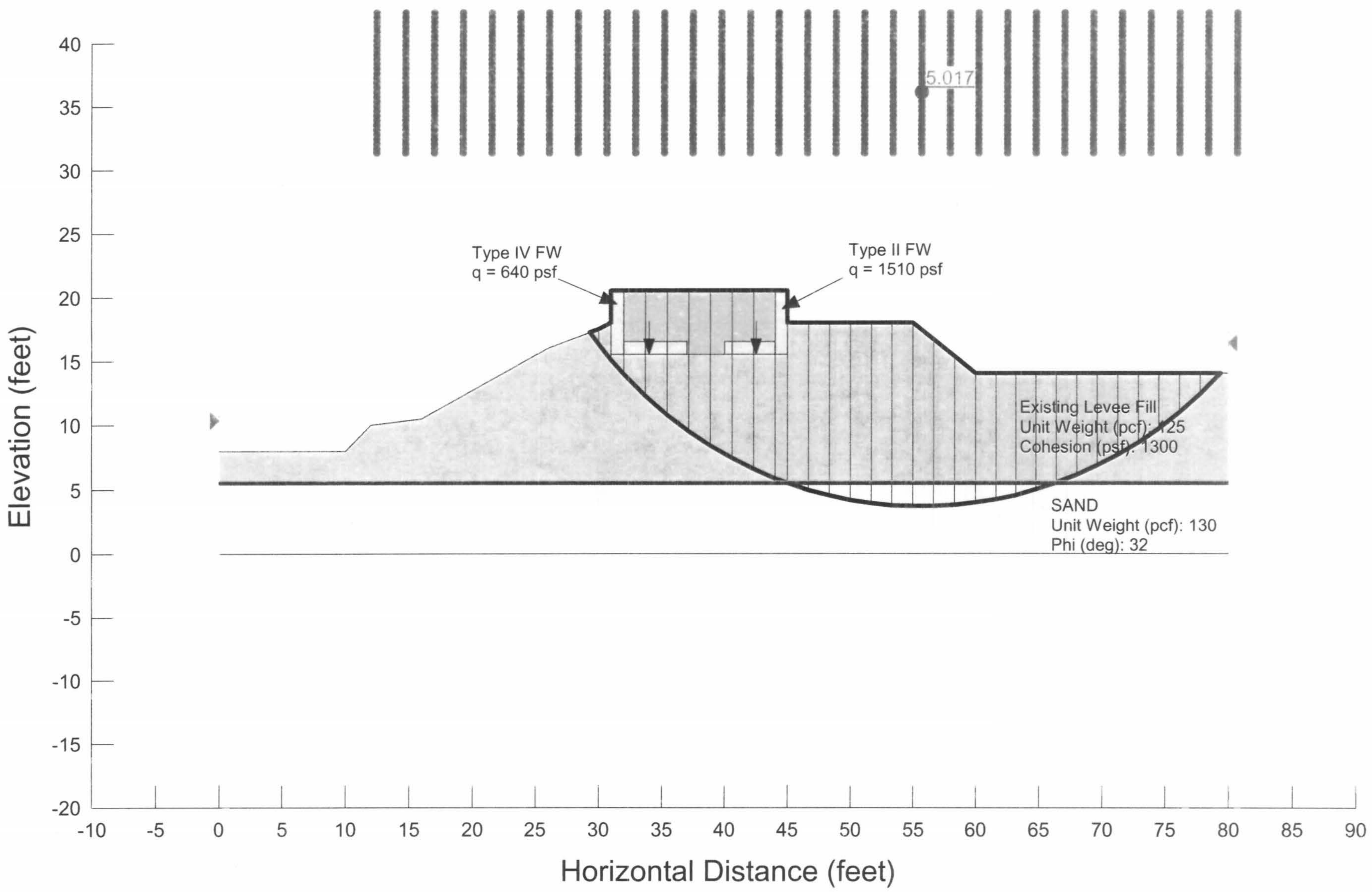
Filename: O:\Geotechnical\Permanente Creek\Slope Stability\WB-1SUDRNr.UT4

Time: 15:18:19

Description: Permanente Creek Flood Protection Project
Comments: Floodwall WB-3 Type II and IV Walls w/Maintenance Road Fill
File Name: WB-3.EOC.slz
Analysis Method: Spencer
Direction of Slip Movement: Right to Left

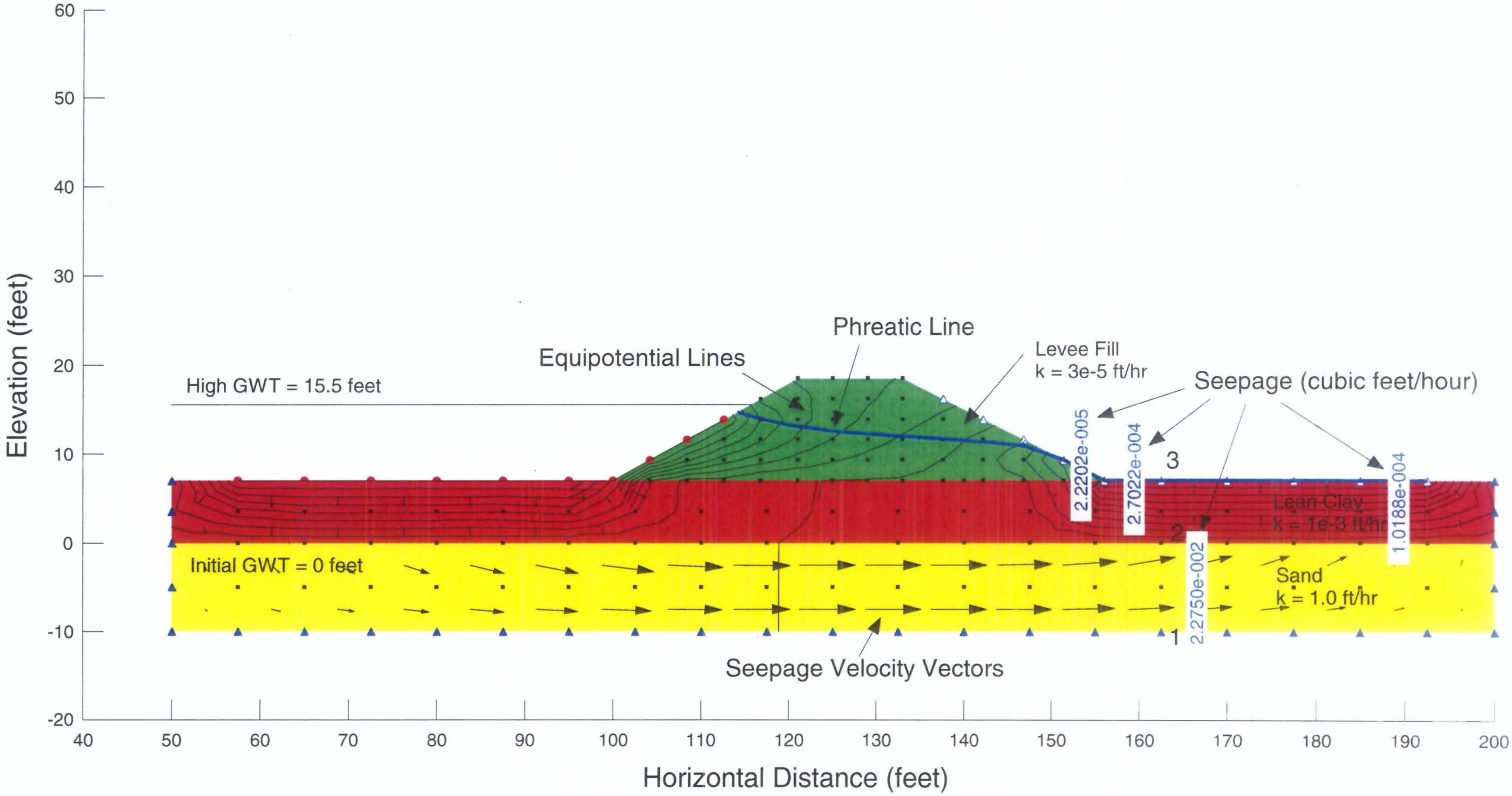


Description: Permanente Creek Flood Protection Project
Comments: Floodwall WB-3 Type II and IV Walls w/Maintenance Road Fill
File Name: WB-3lr.EOC.slz
Analysis Method: Spencer
Direction of Slip Movement: Left to Right

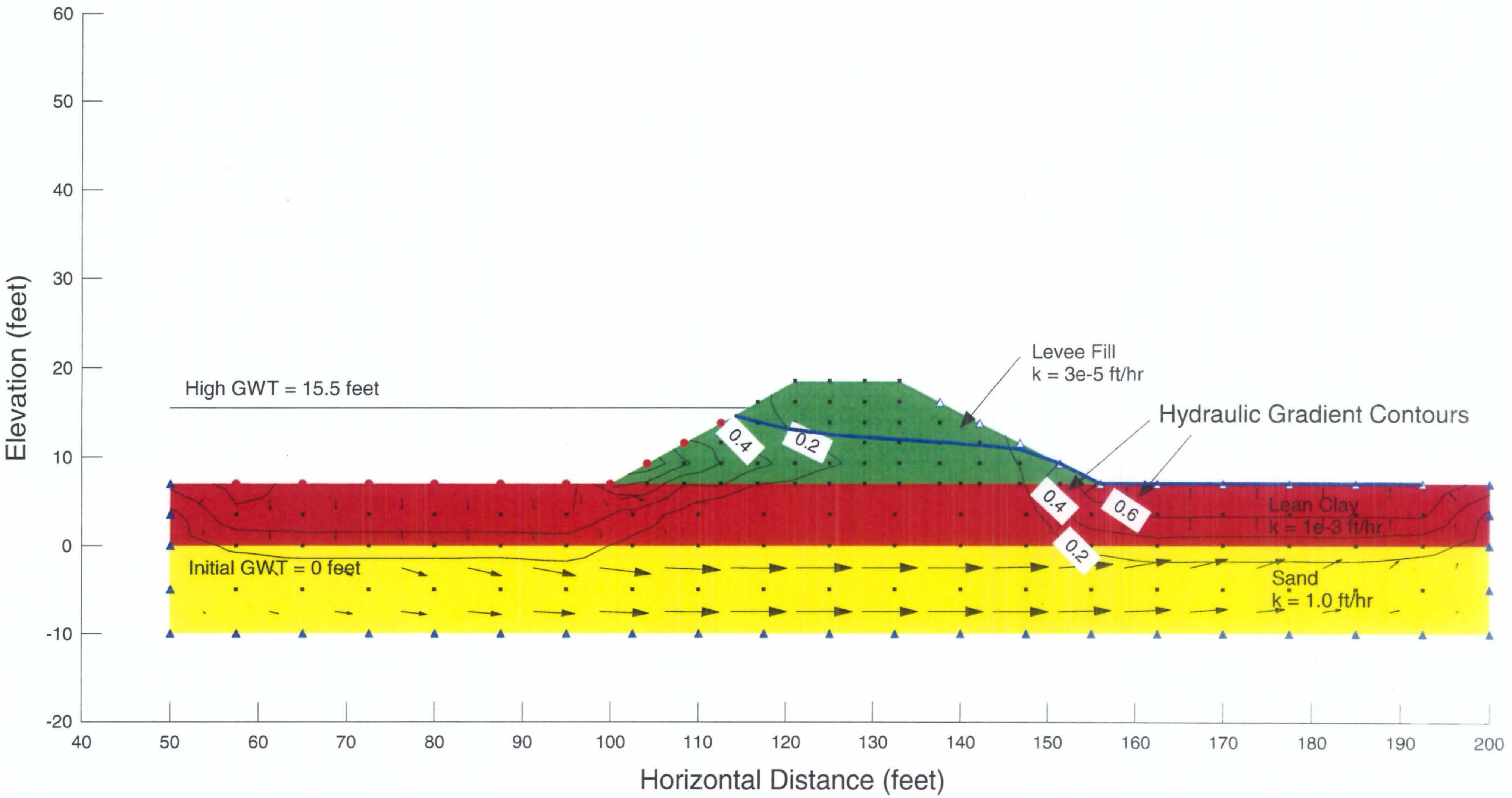


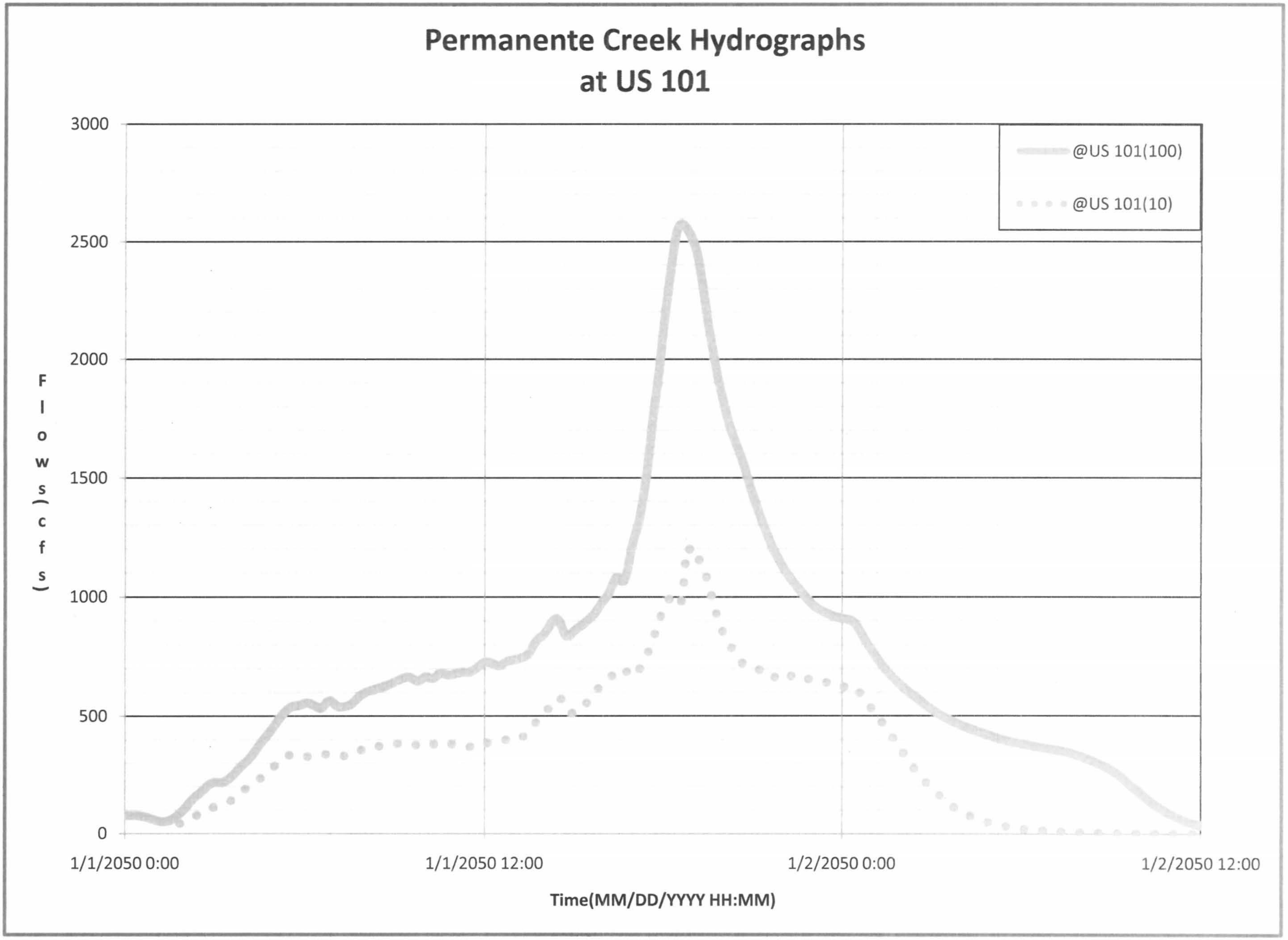
APPENDIX E

Description: Permanente Creek Flood Protection Project
Comments: Levee WB-1 Steady State Seepage
File Name: WB-1.sez
Analysis Type: Steady-State



Description: Permanente Creek Flood Protection Project
Comments: Levee WB-1 Steady State Seepage
File Name: WB-1a.sez
Analysis Type: Steady-State

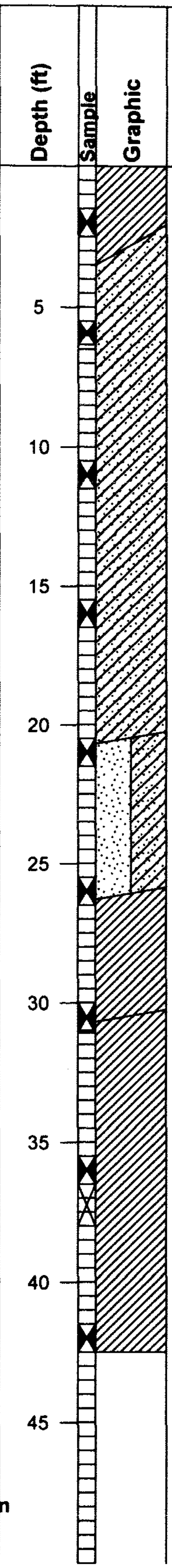



Permanente Creek Hydrographs
at US 101
3000
2500
2000
1500
1000
500
0
Flows (cfs)
1/1/2050 0:00
1/1/2050 12:00
1/2/2050 0:00
1/2/2050 12:00
Time(MM/DD/YYYY HH:MM)
@US 101(100)
@US 101(10)

APPENDIX F

Boring Location Off Mountain View Avenue, Los Altos
Drill Rig CME 75, 8" Hollow Stem Auger
Top Elevation 116.1 Ft. **Date Drilled** 12/4/2003

Laboratory Tests	Blows/Foot	Pocket Pen (tsf)	Torvane (tsf)	Moisture Content (%)	Dry Density (pcf)	Soil Description
						Brown Sandy Lean CLAY (CL) Very stiff to hard, moist (Fill)
UC= 3820	15*	>4.5		14.8	115	
						Reddish Brown Clayey SAND (SC) Medium dense, moist, trace gravel to 1 inch
TxCU$_{(P)}$= 380 (450)	20*	>4.5		14.5	116	
- #200=41.9% LL= 40%; PI= 20%; TxCU$_{(P)}$= 930 (1110)	13*			12.0	119	
TxCU$_{(P)}$= 2440 (3130)	22*			16.8	112	
- #200=9.4%	29*	4.5				Reddish Brown Poorly Graded SAND with Clay and Gravel (SP-SC) Dense, moist
MA: - #4=60.0% - #200=2.5%	31*	4.5				
						Light Brown Lean CLAY with Sand (CL) Hard, moist
- #200=47.9%	23*					
						Brown Sandy Lean CLAY (CL) Very stiff, moist, trace fine gravel
	25* 19	3		21.8	109	Sand lense at 36 feet
	25*	4		16.3	116	Bottom of Boring at 42.5 Feet



*Blows/foot converted to approximate standard penetration test blow counts.

Santa Clara Valley Water District

LOG OF BORING NO. B-4

Permanente Creek Planning Study
Mountain View/Los Altos, CA

PLATE 6

Drawn	Job Number	Approved	Date	Revised	Date
K. Neuman	102401				

LOG OF BORING PERMANENTE CREEK PLANNING STUDY.GPJ SANTA CLARA VALLEY WATER.GDT 3/17/04

APPENDIX G

GEOTECHNICAL INVESTIGATION REPORT
PERMANENTE CREEK LEVEE EVALUATION
SANTA CLARA COUNTY, CALIFORNIA

PARIKH CONSULTANTS, INC.

Practicing in the Geosciences

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GEOTECHNICAL ENGINEERING INVESTIGATION REPORT

PERMANENTE CREEK LEVEE EVALUATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

INTRODUCTION

This report presents the results of our Geotechnical Engineering Investigation performed for evaluating stability of the existing levees along Permanente Creek in the City of Mountain View, California. The Project Location Map, Plate 1, shows the location of the site.

This report presents our understanding of the project, existing site conditions, purpose and scope of investigation followed by description of field exploration, general geology at the site, seismicity, our interpretation of the subsurface and groundwater conditions, and our findings and discussions. The work presented in this report was performed in general accordance with our agreement with Biggs Cardosa Associates, Inc.

PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the site, to determine their engineering properties, to evaluate the condition of the existing levees as per US Army Corps of Engineers guidelines, and to assist Santa Clara Valley Water District (SCVWD) in their certification with FEMA. A detailed scope of work is presented in our proposal to Biggs Cardosa Associates.

The scope of work for this investigation included review of readily available soils and geologic literature pertaining to the site; obtaining representative samples and logging soil materials encountered in twelve exploratory borings; laboratory testing of collected samples, engineering analysis of the field and laboratory data; and preparation of this report.

SITE CONDITIONS

The description of existing conditions at the project site presented in this section of the report is based on our field observations, topographic information provided to us by the Water District, and the available as-built plans. The project site includes the existing levees along Permanente Creek, between Route 101 (Approximate Sta. 38+50) and the Salt Ponds located at approximately Sta. 19+50 (see attached Site Plan, Plate 2). Except for the concrete lined channel near Route 101 and under the bridges, the creek banks are earth slopes which appear to be constructed by raising the old natural creek levees. The existing levees at the site were constructed sometimes in 1960s and have been subsequently modified at most locations. Majority of the modifications include, raising the levees, and backfilling behind the landward slopes of the levees.

The levee heights (measured from the bottom of the creek) generally increases from about 4 m (13 feet), near Route 101, to about 5.5 m (18 feet) towards the downstream-end of the project. The riverside slopes of the levees are inclined from 1V:1.5H (between about Sta. 27+00 to 38+60) to 1V:2H (downstream of Sta. 27+00). The landside slope of the levees have been considerably modified since 1960s. They have been partially or completely backfilled during construction of adjoining industrial parks, landfills, golf courses, etc. Based on our review of the as-built levee sections, shear keys are provided under the levees north of approximate Sta. 32+00 (downstream from Amphitheater Parkway). These keys are usually 2.5 m to 3 m (8 feet to 10 feet) deep and about 3 m wide. A 3.5 m to 4 m (11.5 feet to 13 feet) wide maintenance road has been constructed on top of the levees. The levee slopes are presently covered with sparse to dense vegetation including grass, shrubs, trees and pickel-weeds. Excessive settlements, slope failures, cracking, etc. were not observed within the levees. However, localized sloughing, erosion, etc. were observed at many locations.

GEOLOGY

The site is located along the northern margin of the Santa Clara Valley, approximately 3 km (1.9 miles) west of the San Francisco Bay. The general geology at the site and its vicinity was mainly studied from a 1:62500 scale geologic map of late cenozoic deposits compiled by Helley and Brabb (1971). A portion of this map showing the site and its vicinity is presented on Geologic Map, Plate 3. Based on the physical properties and relative age of the deposits, Helley and Brabb have mapped unconsolidated, cenozoic deposits at the site and its surroundings. The surficial geologic units in the site vicinity were deposited during Tertiary to Quaternary Periods. Relatively younger units, i.e. Interfluvial Basin Deposits (Qb), Fluvial Deposits (Qyfo), Young Alluvial Fan Deposits (Qyf) and Older Fan Deposits (Qof) which mainly consist of alluvial, marine and lacustrine deposits are mapped within open valley and narrow canyons. Stream bed deposits and alluvial aprons and fans of earlier period (Qof) mapped in the site vicinity grade baywards into the Merritt Sand (Qm) along the margin of the bay. The younger fan deposits are superimposed on the older fan systems and grade into the Older Bay muds (Qobm) of the San Francisco Bay along the bay margin.

On a regional basis, the native soils underlying the site chiefly consist of silts and clays deposited in a bay-margin environment. Specifically, at the project site, the native surficial deposits have been mapped as Holocene Interfluvial Basin Deposits (Qb) and Pleistocene Older San Francisco Bay Muds (Qobm). Helley and Brabb have described these units as follows:

> "INTER FLUVIAL BASIN DEPOSITS, Qb - Mainly organic rich clay and silty clay that locally contain modern fresh-water Gastropods and Palecypods."
>
> "OLDER SAN FRANCISCO BAY MUD, Qobm - semi consolidated organic-rich clay deposits; underlies younger San Francisco bay mud and young alluvial fan deposits; extending landward to maximum elevation of 10 feet above Mean Sea Level."

The native surficial deposits at the site are exposed in the creek bed. At other locations, they are generally covered by asphalt concrete or artificial fill.

The bedrock at depth in the project area is presumed to be the Franciscan Complex of the upper Jurassic to Cretaceous age. The bedrock in the area is believed to be overlain by more than 150 m (500 feet) thick deposits (Rogers and Williams, 1974) of Tertiary marine/non-marine sediments and by Pleistocene to Recent deposits.

FIELD EXPLORATION

Based on the plans provided, discussions with client, and readily available geologic and existing boring information from previous studies, twelve borings were drilled up to a depth of 12.7 m (41.5 feet) below the top of the levees. The attached Plate 2, Site Plan, shows the approximate locations of these borings.

The test borings were advanced with a truck-mounted drill rig using a 208-mm (8 inches) diameter hollow stem auger. Selected drive samples were obtained from the borings at various depths using a 64 mm (2.5 inches) I.D. Modified California Sampler or a 36 mm (1.375 inches) I.D. Standard Penetration Sampler. The sampler was driven into the subsurface soils under the impact of a 64 kg (140 lbs) hammer having a free fall of 76 cm (30 inches). The blow counts required to drive the sampler for the last 30 cm (12 inches) are presented on the Log of Test Borings, Appendix A. (When correlating standard penetration data, the blow counts for the Modified California Sampler can be taken as roughly twice that for the Standard Penetration Test in similar soils). Pocket penetration tests were also performed on clay samples during field investigation to evaluate their consistency. The soil samples obtained during drilling were visually classified in the field and then transported to our laboratory for further evaluation and testing. All the boreholes were grouted as per SCVWD guidelines.

Laboratory tests including moisture content, dry density, atterberg limits, sieve analysis, consolidation, unconfined compression, and triaxial (Consolidated Undrained test with pore pressure measurements) were performed on soil samples collected during field exploration to determine the physical and engineering properties of the subsurface soils. Test methods and test results are presented on plates attached in Appendix B. Laboratory test results for moisture content, dry density, sieve analysis, atterberg limits and unconfined compression tests are presented on the attached boring logs, Appendix A.

The description of the soils encountered and relevant boring information are presented on the boring logs, attached in Appendix A. The bore logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs. The abrupt stratum changes shown on these logs may be gradual and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations.

To supplement our study, we also made reference to geotechnical investigation reports prepared by Lowney Associates (dated 3/17/99). Relevant boring logs from this report are attached in Appendix D.

SOIL CONDITIONS

Based on the information obtained from our study of the available geotechnical information; and, data obtained from our exploratory borings and laboratory tests, the site is generally underlain by levee fills overlying native alluvial/marine deposits predominantly consisting of clays.

Fills, used to construct the flood control levees were encountered in the borings. These fills extend to depths of about 4 m to 9 m (13 feet to 29.5 feet) (where shear keys are provided) below the top of the levees. The fills appear to be well compacted. The fill material is generally stiff to very stiff, lean and fat clays containing some sand. Scattered lenses of compacted clayey and silty sands were also encountered within the levees (e.g. Sta. 30+10 and Sta. 26+55).

Below the fills, the borings generally encountered low to highly plastic clay layers interbedded with sand lenses/layers. The interbedded sand layers generally diminish downstream from approximate Sta. 25+00. Organic clays, locally known as bay mud were encountered under the levee fill in borings drilled downstream of approximate Sta. 24+50.

The clay layers underlying the site predominantly consist of lean clays (CL) containing varying amounts of fine sands. The Plasticity Indices (PI) and Liquid Limits (LL) of these clays usually vary between 9 to 31 percent and 29 to 49 percent, respectively. These clays are medium to very stiff and have natural water content and total unit weights in the range of 10 to 25 percent and 18 kN/m^3 to 19.7 kN/m^3 (115 pcf to 125 pcf), respectively. Most of the lean clay layers encountered in our borings appear to be slightly to moderately overconsolidated. The overconsolidation appears to be due to drying and desiccation of the clays just after they were deposited.

Discontinuous, fat clay (CH) layers were encountered at some locations, between the levee fill and the underlying lean clay layers. These fat clay layers are usually about 1.5 m to 3 m (5 feet to 10 feet) thick. They usually contain some sand, and are generally stiff to very stiff in consistency. Based on the Atterberg limits test data, these fat clays are characterized as having medium to high potential for expansion. Fat clays are very fine grained and are deposited in calm environments such as basins. The fat clays at the site appear to be moderately overconsolidated.

An about 1.5 m to 3 m (5 feet to 10 feet) thick layer of organic clays (locally known as "Bay Mud") was encountered in the borings drilled downstream of approximate Sta. 24+50. This layer was encountered between the levee fill and the underlying lean clay layers. The bay mud encountered in our borings is medium stiff and usually has a natural moisture content range of 50 to 70 percent. Total unit weights of these organic clays generally range from 14 to 15 kN/m^3 (89 to 95 pcf). These organic clays were deposited in a marine environment and may contain shells and scattered peat. Based on the laboratory test results, the bay mud at the site appears to be slightly overconsolidated.

Sand layers/lenses were mostly encountered upstream of approximate Sta. 25+00. The sand layers underlying the site are mostly discontinuous, randomly scattered and are usually about 1.5 to 3 m (5 feet to 10 feet) thick. The sands encountered in our borings are generally fine grained and contain substantial non-plastic fines (silts). They are generally medium dense.

Groundwater was encountered in our borings at depths ranging from 3.5 m to 6 m (11.5 feet to 21 feet). We expect the groundwater at the site to be at or near the creek bottom. It should be noted that the groundwater level at the site may change with passage of time due to groundwater fluctuations from season to season, weather conditions, and other factors which may not have been present at the time of the investigation.

Detailed descriptions of the materials encountered in the exploratory borings are presented on boring logs attached in Appendix A. The descriptions and related information presented on these logs of test borings depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Subsurface conditions at other locations may differ from conditions occurring at the locations explored. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

EARTHQUAKE CONSIDERATIONS

The project site is located in a seismically active part of northern California. Many faults exist in the San Francisco Bay Area which are capable of producing earthquakes which may cause strong ground shaking at the site. Attached Plate 4, Fault Map presents the locations of the fault systems relative to the project site.

Maximum credible earthquake is defined as the maximum earthquake that appears capable of occurring under the known tectonic frame. Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (1996) are summarized in Table 1. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

TABLE 1

Fault	Distance from Site (km)	Maximum Credible Earthquake Magnitude
San Andreas	14.5	8
Hayward	15.0	7½
Calaveras	22.0	7½
Monte Vista West	10.5	6½
Monte Vista East	4.5	6½

Potential seismic hazards at a site may arise from these sources: surface fault rupture, strong ground shaking and liquefaction. No known active or potentially active faults cross the project site. Therefore, in our opinion, the potential for ground surface rupture due to faulting at the site is considered low.

As shown on Plate 4, Fault Map, active faults including San Andreas Fault, Hayward Fault, Monte Vista East, Monte Vista West and Calaveras Fault are located in the vicinity of the site.

A major earthquake on these faults is expected to produce very strong ground shaking at the site. Based on the map prepared by Mualchin (1996) a peak bedrock acceleration of about 0.4g is anticipated at the site. The corresponding peak ground acceleration may be on the order of 0.3g.

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless silts and sands of low relative density are the type of soils which usually are susceptible to liquefaction. Clays are generally not susceptible to liquefaction.

Analyses of the liquefaction potential of isolated sand deposits underlying the site were performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7½ earthquake on the nearby Hayward Fault resulting in a peak ground acceleration of about 0.3g. This method compares estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. Cohesionless soils below a depth of 15 m (50 feet) and cohesionless soils above the groundwater level were considered to have low liquefaction potential. Based on our analyses, many of the shallower, discontinuous medium dense sand and silty sand deposits underlying the site may liquefy during a strong earthquake. The extent and consequences of this liquefaction was further studied from historical data compiled by Youd and Hoose (1978). Youd and Hoose have documented evidence of liquefaction related phenomenon in the site vicinity during the California Earthquake of April 18, 1906 on the nearby San Andreas Fault. No liquefaction site has been identified by them in the project vicinity. No evidence of liquefaction are documented in the immediate site vicinity during the 1906 earthquake or the recent Loma Prieta Earthquake (1989).

Based on historical evidence, the present topography at the site and considering that the soils which overlie the potentially liquefiable soils are predominantly cohesive in nature, in our opinion, the consequences of liquefaction in the area could be limited to some post-liquefaction settlements of the ground surface. It is estimated that the post-liquefaction settlements may be on the order of 10 mm to 35 mm (0.375 inches to 1.375 inches), and probably would be random and localized.

FINDINGS AND DISCUSSIONS

Evaluation of Existing Levees

The existing levee sections were analyzed for seepage and slope stability as per guidelines outlined in US Army Corps of Engineer's Levee design and construction manual EM 1110-2-1913. The results of these analyses are discussed in the following subsections.

Seepage Analysis

Analyses for seepage through- and under- the levee were conducted. The existing levee sections used in the analysis are shown on plates attached in Appendix C.

The existing levee fills and the underlying native soils are predominantly lean and sandy lean clays. Therefore, the levee materials and the underlying soils were modeled as clays for seepage analysis. Laboratory permeability tests were not performed on these fine grained soils and their permeabilities were estimated from other laboratory test results and readily available empirical relationships.

Seepage analyses were performed using a finite element computer program, SEEP2D which was developed at Brigham Young University. The design maximum probable flood level for seepage analysis was taken to be about 1.5 m (5 feet) below the top of the existing and modified levees. Since the landside slopes of the levees have been backfilled, seepage at the site is mainly in the form of percolation towards the groundwater table located just below or at the bottom of the creek. Based on these results and based on the duration of the probable maximum flood event (3 to 4 days), it is anticipated that the seepage across the levees will be negligible and the levee materials are not expected to get saturated during these probable maximum flood events. The studies indicate that seepage control measures may not be required. The phreatic surfaces estimated during seepage analysis are as shown on plates attached in Appendix C. These phreatic surfaces were further used for slope stability analyses.

Slope Stability Analysis

Slope stability analyses were performed on the existing levee sections using Bishop's method and the computer program UTEXAS3 developed by Stephen Wright of the University of Texas at Austin. Computer program XSTABL, was also used for some slope stability conditions and to verify results obtained from UTEXAS3.

Critical sections used for the slope stability analyses were developed based on the existing and proposed slope geometries, geology, field observations, boring logs and laboratory test results.

In general, these sections are to be analyzed for Cases I to V outlined in the US Army Corps of Engineer's Manual EM 1110-2-1913. These critical conditions and the corresponding minimum strength and factors of safety requirements are presented on Table 2.

TABLE 2

CASE NO.	DESIGN CONDITION	SLOPE ANALYZED	SHEAR STRENGTH	MINIMUM FACTOR OF SAFETY
I	End of Construction	Creekside & Landside	Q or S	1.3
II	Sudden Drawdown	Creekside	S where < R R where < S	1.0
III	Intermediate River stage	Creekside	S where < R (R+S)/2 where R < S	1.4
IV	Steady seepage from full flood stage	Landside	S where < R (R+S)/2 where R < S	1.4
V	Earthquake: Case I, or III with seismic loading	Creekside and Landside	Seismic Coefficient 0.2g	1.0

In our opinion, the excess pore water pressures under the existing levees have dissipated after its modifications and therefore the End of Construction case (Case I) was not analyzed for the existing levee sections. However, we have analyzed the levees for existing conditions using total stress analysis. The landside of the levee slopes have been backfilled along most of the alignment. Therefore, stability for Case IV is not applicable. Only creekside slope stability was evaluated for this study i.e. Cases II, III and V.

The strength parameters used in the analysis are presented on Plates C-1 to C-12. These strength parameters were estimated from field Standard Penetration Test results, pocket penetrometer test results, unconfined compression test results, and triaxial consolidated undrained tests with pore water measurements (Appendix B). Where the triaxial test results showed some scatter, the effective and total strength envelopes used in the analysis were conservatively estimated based on the test results, engineering judgement and published empirical relations.

Tension cracks were assumed in the upper 1.5 m (5 feet) of the levee top and side slopes. The tension cracks were assumed to be filled with 0.9 m (3 feet) of water. The maintenance road on top of the levee is presently being used. The load of the levee due to the traffic on the top was modeled as an uniform surcharge of 12 kPa (250 psf).

Earthquake loading conditions were modeled by using a seismic loading coefficient of 0.2g for pseudo-static analysis. The shear strengths, unit weights, geometry, phreatic surfaces used in the analyses and the factors of safety and possible critical sliding surfaces obtained from slope stability analyses are presented on Plates C-1 to C-12. The factor of safety for Case III i.e. Intermediate River Stage is the minimum obtained by varying the water within the creek level from creek bottom to the maximum design flood level. Due to relatively long duration of creek levels expected for this case, the groundwater was assumed to rise to the creek water level for this analysis.

The factors of safety obtained from the analyses are summarized on Table 3. Based on these results, it is concluded that the existing and modified levees proposed for the project have adequate stability under static and seismic loading conditions.

The stability analyses evaluates the global slope stability. Minor sloughing, cracking, etc. should be anticipated due to clayey nature of the levee fill. Rodent burrows, etc. were also observed during our site visits. Existing and future sloughing, cracking, gullying, erosion, etc. should be addressed as a part of regular maintenance.

TABLE 3

Location (Riverside slopes)	Factors of Safety for Existing Levee Sections							
	Static Cases				Seismic Cases (0.2g)			
	Existing	**II**	**III**	**IV**	**Existing**	**II**	**III**	**IV**
Sta. 32+00 to Sta. 38+70	5.5	1.6	1.9	NA	3.7	1.13	1.3	NA
Sta. 26+50 to Sta. 32+00	4.4	1.8	2.4	NA	2.8	1.2	1.5	NA
Sta. 23+00 to Sta. 26+50	4.4	2.1	2.0	NA	2.8	1.5	1.4	NA
Sta. 19+50 to Sta. 23+00	2.9	1.6	1.5	NA	1.9	1.04	1.0	NA

We understand that the creek capacities may be increased in the future by raising the levees, constructing floodwalls, etc. We believe that this could adversely impact the levee stability, especially downstream of Sta. 24+50, where the soils are weaker than the remaining alignment. Levee stability should be re-evaluated if the levees are planned to be modified in the future.

INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our field exploration and the assumption that the soil conditions do not deviate from observed conditions. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

The findings in this report are valid as of the present date. However, changes in the soil conditions can occur with the passage of time , whether they be due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Very truly yours,
PARIKH CONSULTANTS, INC.

Hemang Desai, P.E., C57931
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

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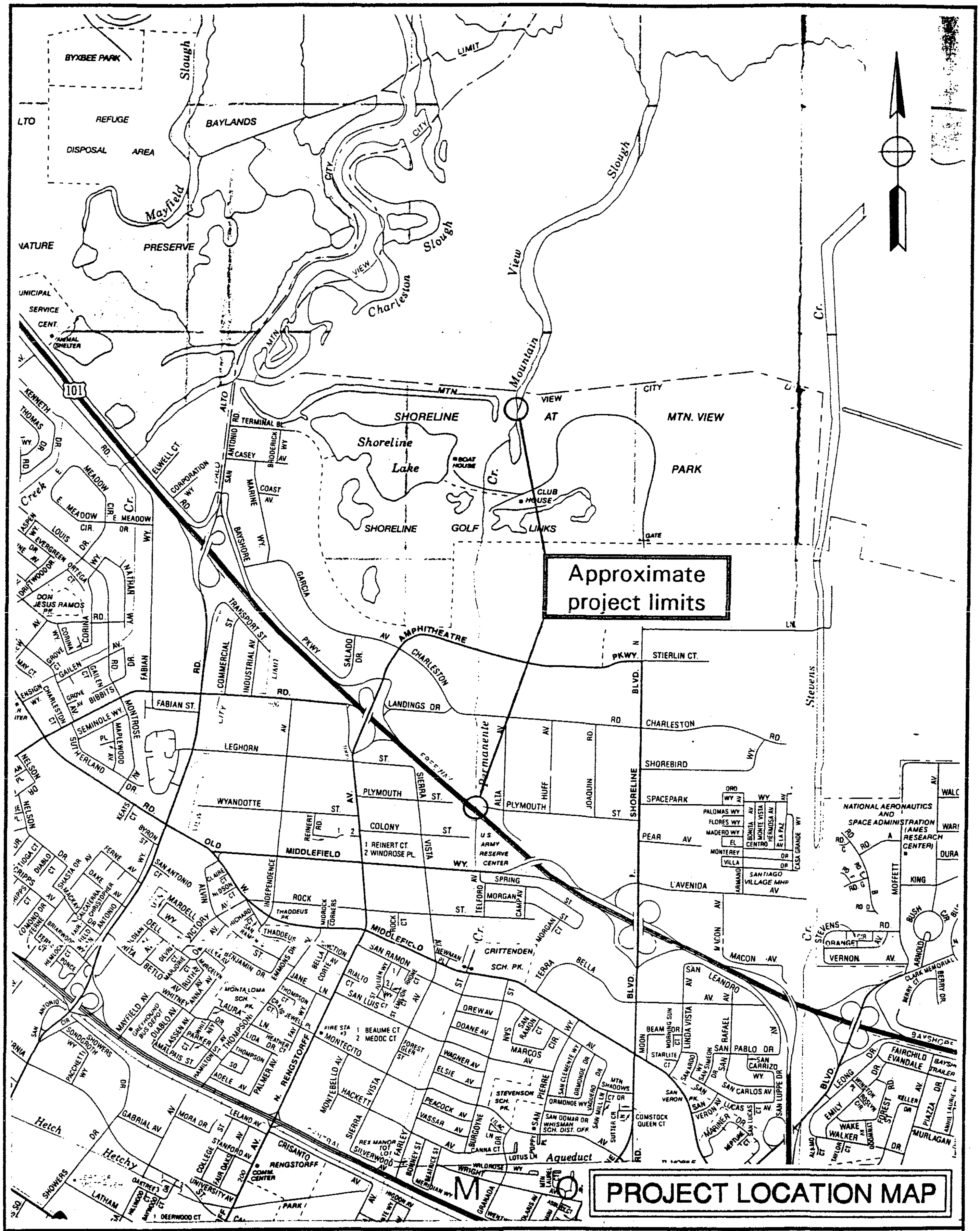
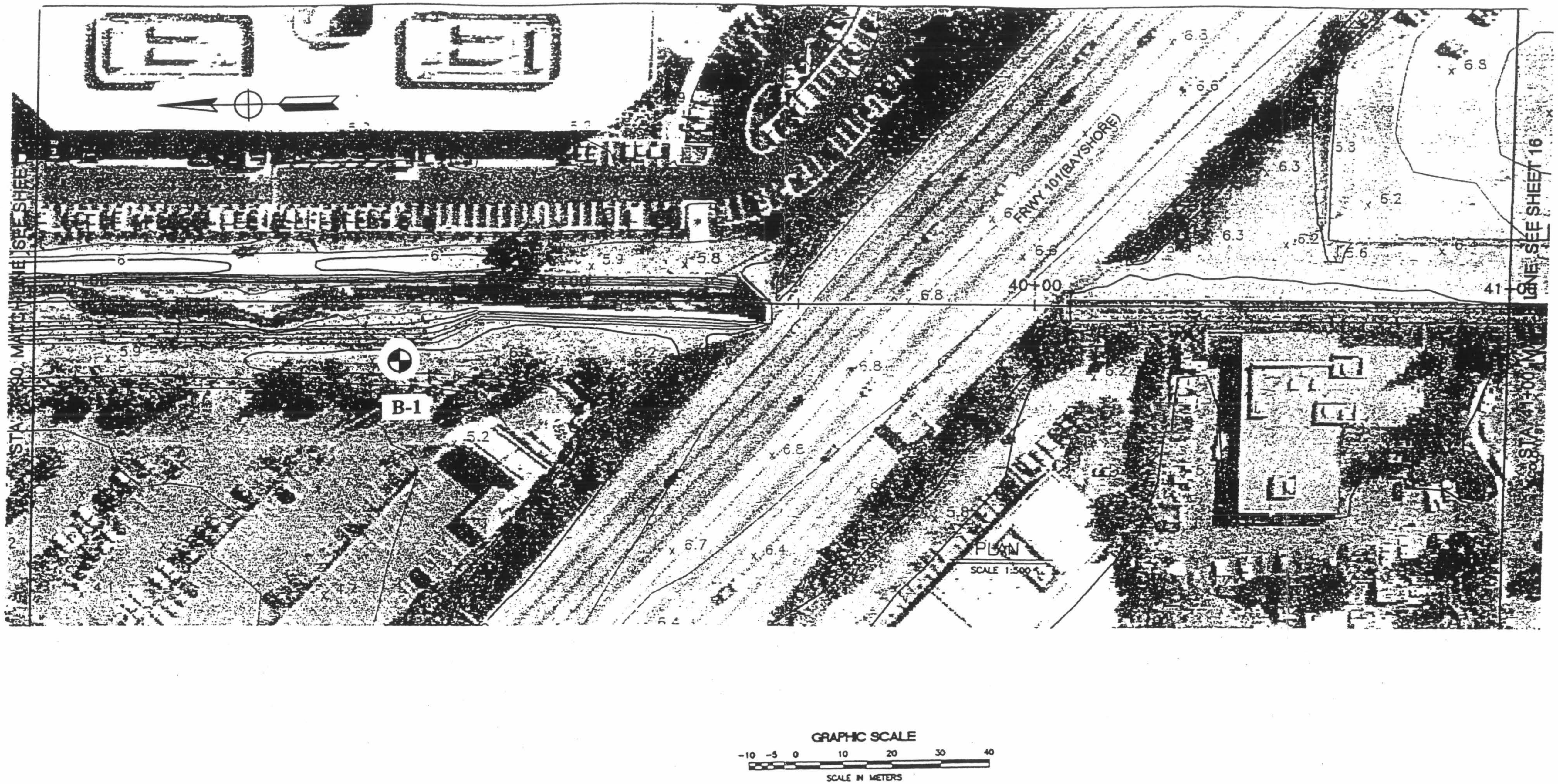


PROJECT LOCATION MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10	PLATE NO: 1



SCALE: 1:1,000

SITE PLAN

SOURCE: Santa Clara Valley Water District, Map and General Plan for Permanente Creek and Hale Creek, San Francisco Bay to Foothill Expressway, Dated 3/12/99

LEGEND:

⊕ Approx. Boring Location

PARIKH CONSULTANTS, INC.
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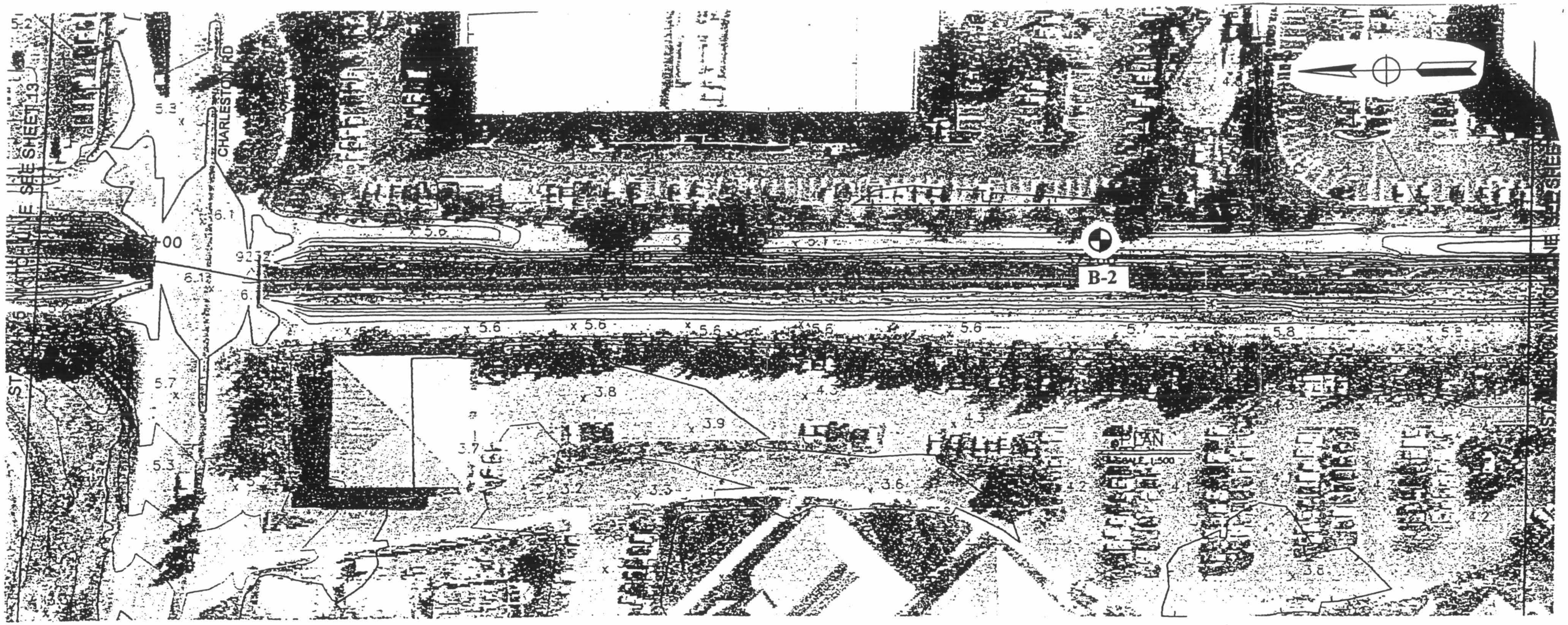
MATERIALS TESTING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: 2A



GRAPHIC SCALE

-10 -5 0 10 20 30 40

SCALE IN METERS

SCALE: 1:1,000

SITE PLAN

SOURCE: Santa Clara Valley Water District, Map and General Plan for Permanente Creek and Hale Creek, San Francisco Bay to Foothill Expressway, Dated 3/12/99

LEGEND:

⊕ Approx. Boring Location

PARIKH CONSULTANTS, INC.
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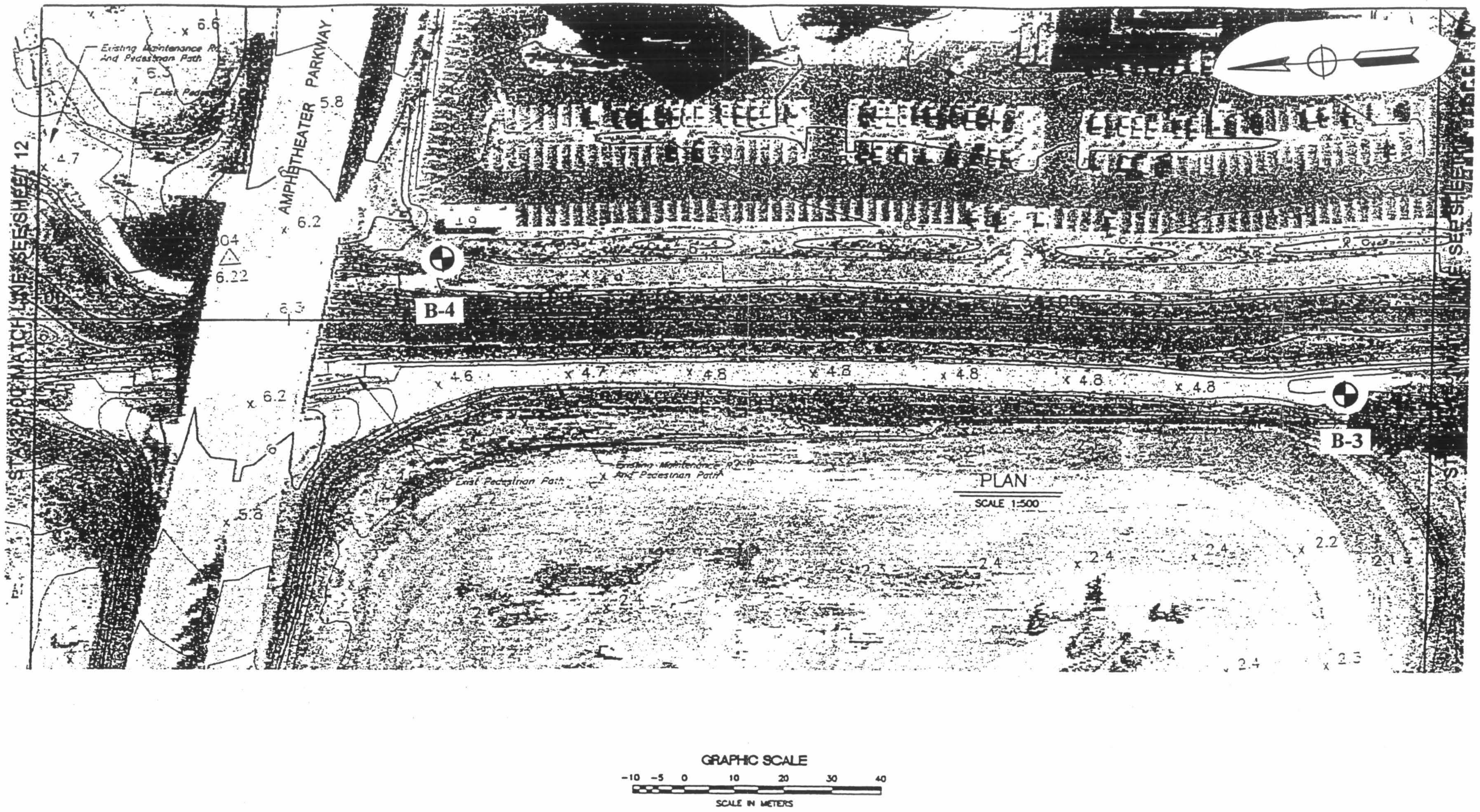
MATERIALS TESTING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: 2B



SCALE: 1:1,000

SITE PLAN

SOURCE: Santa Clara Valley Water District, Map and General Plan for Permanente Creek and Hale Creek, San Francisco Bay to Foothill Expressway, Dated 3/12/99

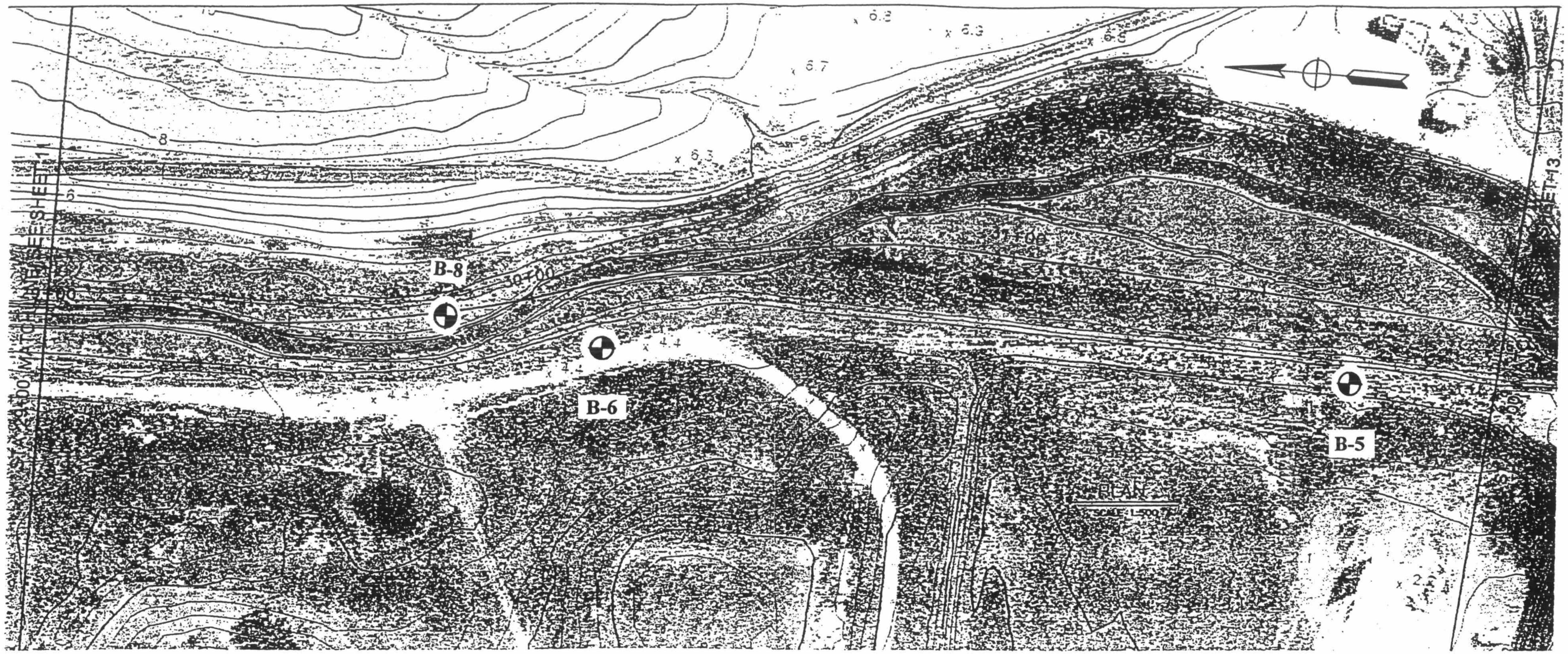
LEGEND:

⊕ Approx. Boring Location

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10	PLATE NO.: 2C



GRAPHIC SCALE

-10 -5 0 10 20 30 40

SCALE IN METERS

SCALE: 1:1,000

SITE PLAN

SOURCE: Santa Clara Valley Water District, Map and General Plan for Permanente Creek and Hale Creek, San Francisco Bay to Foothill Expressway, Dated 3/12/99

LEGEND:

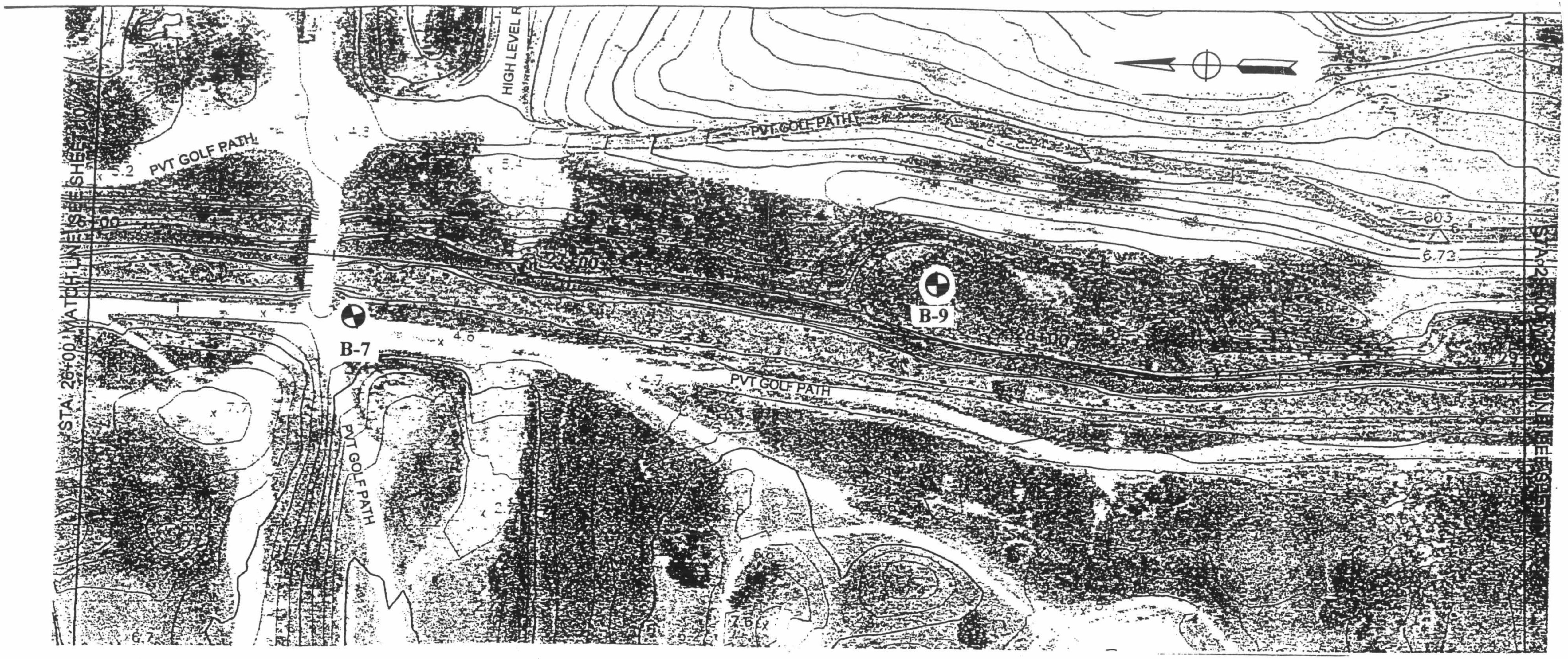
⊕ Approx. Boring Location

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: 2D



GRAPHIC SCALE

-10 -5 0 10 20 30 40

SCALE IN METERS

SCALE: 1:1,000

SITE PLAN

SOURCE: Santa Clara Valley Water District, Map and General Plan for Permanente Creek and Hale Creek, San Francisco Bay to Foothill Expressway, Dated 3/12/99

LEGEND:

⊕ Approx. Boring Location

PARIKH CONSULTANTS, INC.
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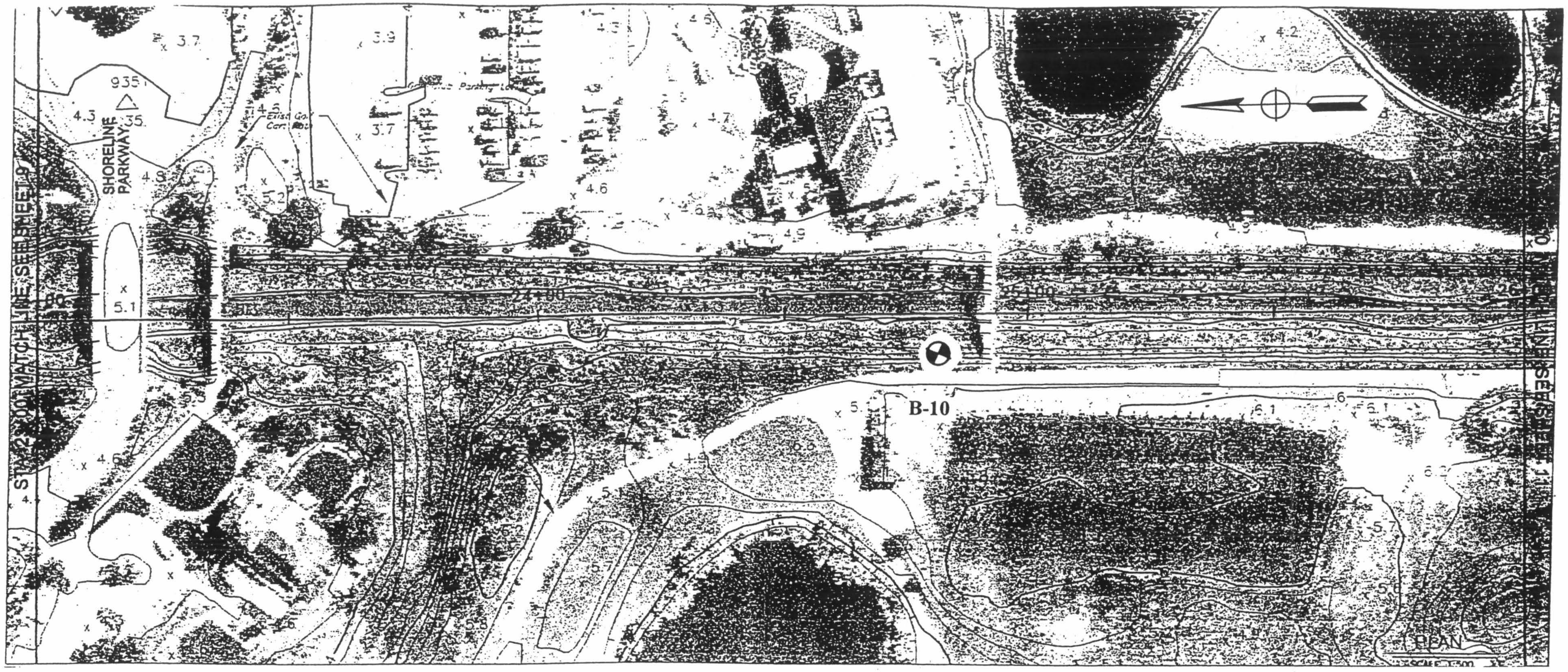
MATERIALS TESTING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: 2E



GRAPHIC SCALE

-10 -5 0 10 20 30 40

SCALE IN METERS

SCALE: 1:1,000

SITE PLAN

SOURCE: Santa Clara Valley Water District, Map and General Plan for Permanente Creek and Hale Creek, San Francisco Bay to Foothill Expressway, Dated 3/12/99

LEGEND:

⊕ Approx. Boring Location

PARIKH CONSULTANTS, INC.
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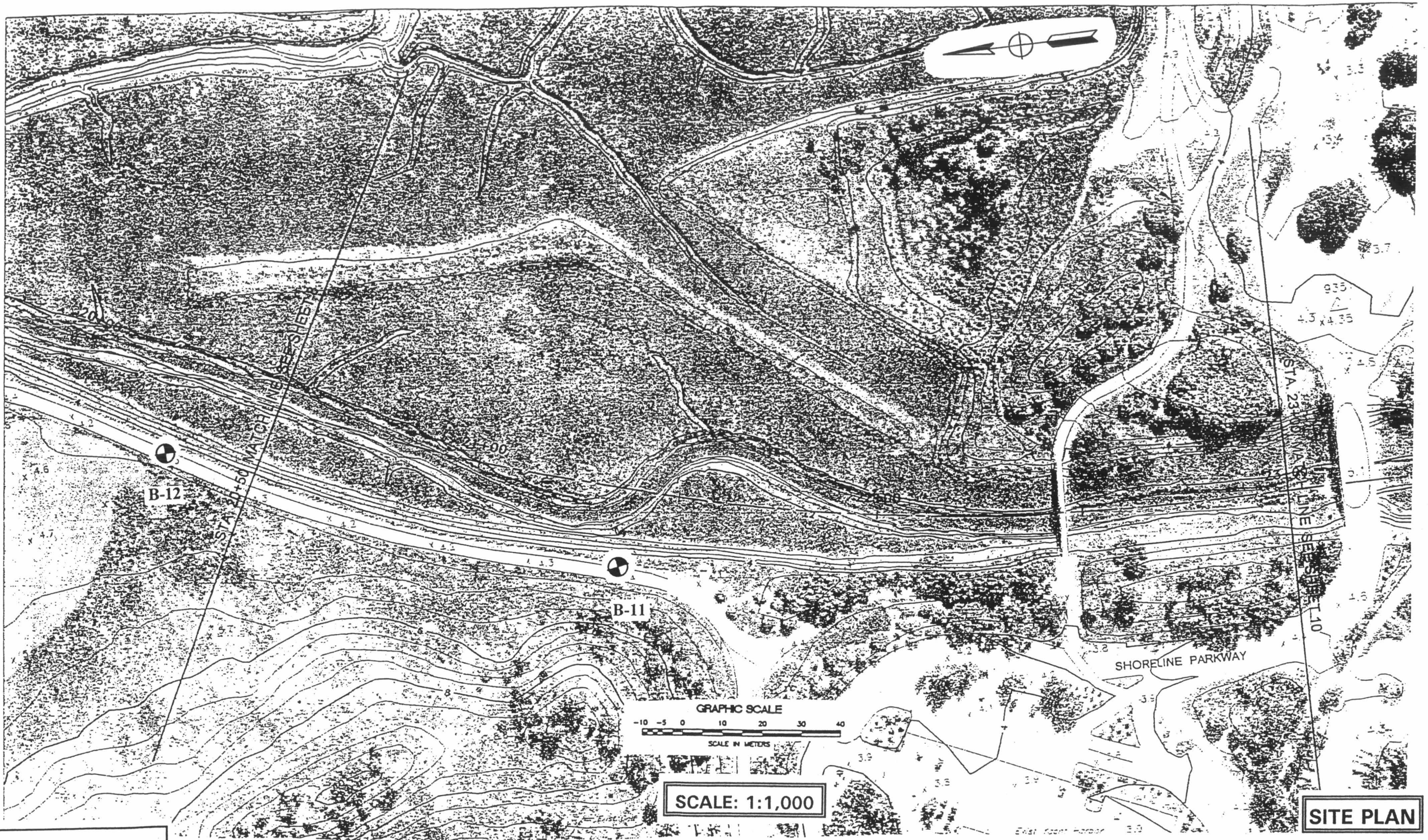
MATERIALS TESTING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: 2F



SCALE: 1:1,000

SITE PLAN

SOURCE: Santa Clara Valley Water District, Map and General Plan for Permanente Creek and Hale Creek, San Francisco Bay to Foothill Expressway, Dated 3/12/99

LEGEND:

● Approx. Boring Location

PARIKH CONSULTANTS, INC.
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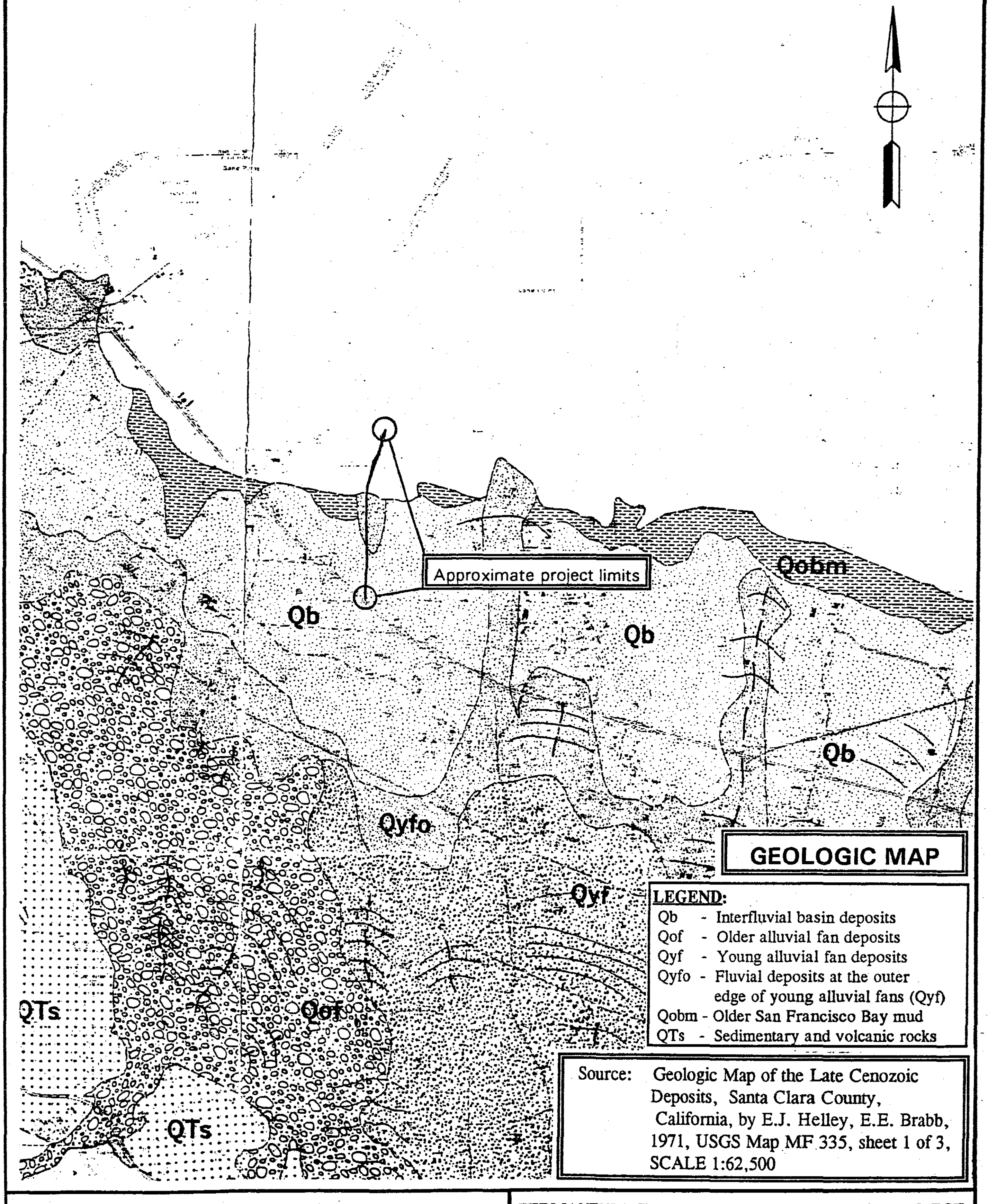
MATERIALS TESTING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: 2G



GEOLOGIC MAP

LEGEND:

Qb - Interfluvial basin deposits
Qof - Older alluvial fan deposits
Qyf - Young alluvial fan deposits
Qyfo - Fluvial deposits at the outer edge of young alluvial fans (Qyf)
Qobm - Older San Francisco Bay mud
QTs - Sedimentary and volcanic rocks

Source: Geologic Map of the Late Cenozoic Deposits, Santa Clara County, California, by E.J. Helley, E.E. Brabb, 1971, USGS Map MF 335, sheet 1 of 3, SCALE 1:62,500

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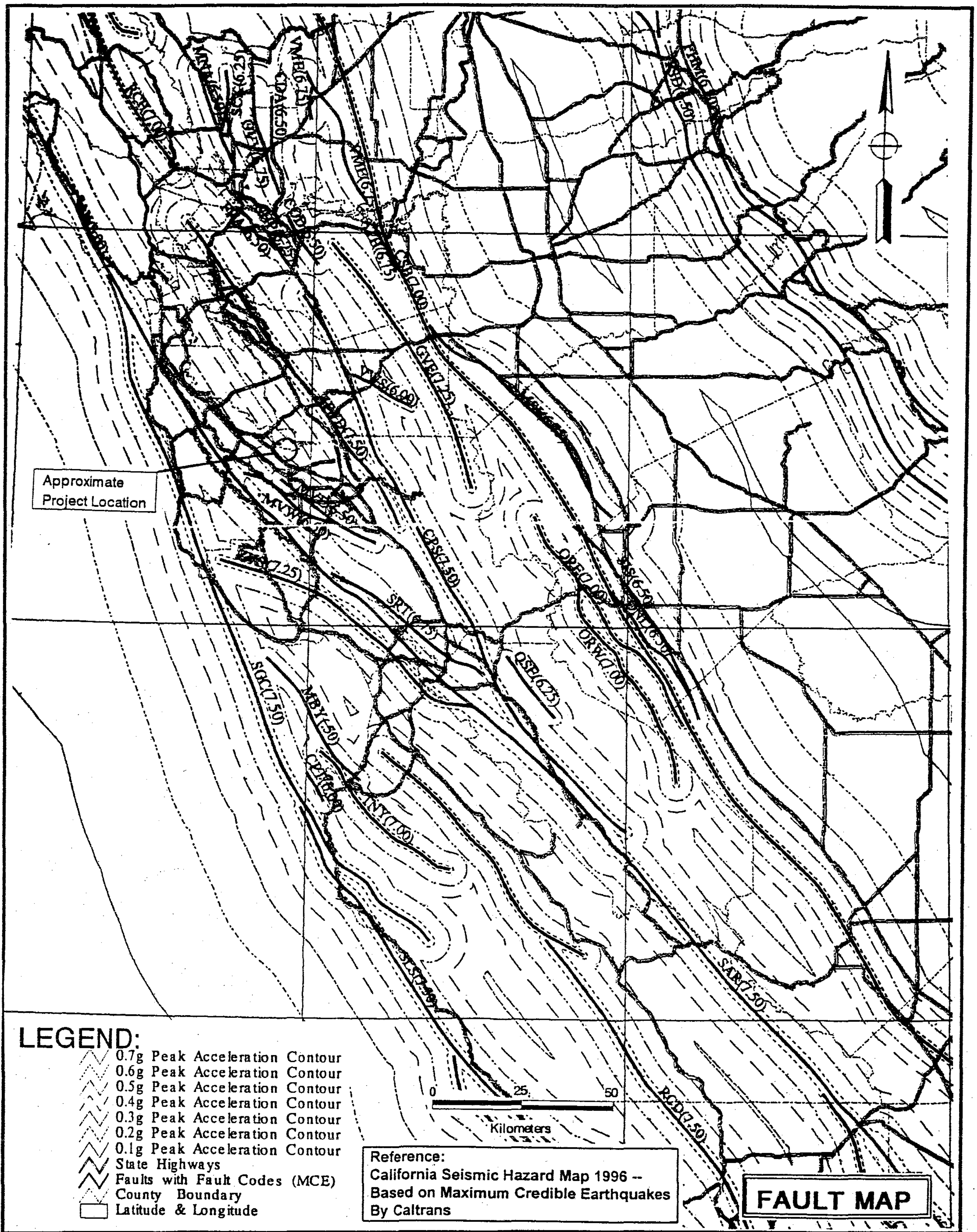
PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10 | **PLATE NO: 3**


Approximate Project Location
GVE(7.25)
CSB(7.00)
CPS(7.50)
SRT(6.75)
SGC(7.50)
MBY(.50)
INY(7.00)
QSB(6.25)
ORW(7.00)
ORE(7.00)
SAR(7.50)
RCD(7.50)
MVW
0
25
50
Kilometers
LEGEND:
0.7g Peak Acceleration Contour
0.6g Peak Acceleration Contour
0.5g Peak Acceleration Contour
0.4g Peak Acceleration Contour
0.3g Peak Acceleration Contour
0.2g Peak Acceleration Contour
0.1g Peak Acceleration Contour
State Highways
Faults with Fault Codes (MCE)
County Boundary
Latitude & Longitude

Reference:
California Seismic Hazard Map 1996 --
Based on Maximum Credible Earthquakes
By Caltrans

FAULT MAP

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PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10	PLATE NO: 4

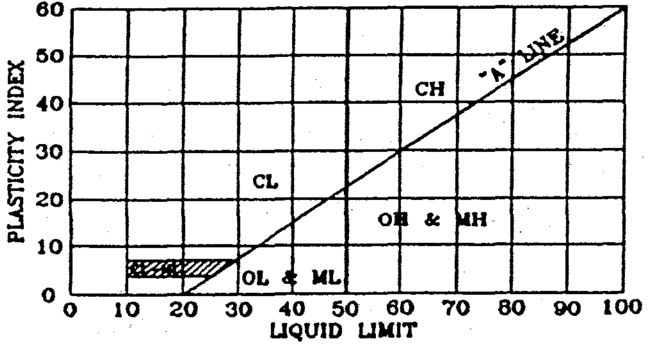
APPENDIX A

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS Less than 5% fines	GW	Well graded gravel, Well graded gravel with sand
			GP	Poorly graded gravel, Poorly graded gravel with sand
		GRAVELS WITH FINES More than 12% fines	GM	Silty gravel, Silty gravel with sand
			GC	Clayey gravel, Clayey gravel with sand
	SANDS 50% or more of coarse fraction passing No. 4 sieve	CLEAN SANDS Less than 5% fines	SW	Well graded sand, Well graded sand with gravel
			SP	Poorly graded sand, Poorly graded sand with gravel
		SANDS WITH FINES More than 12% fines	SM	Silty sand, Silty sand with gravel
			SC	Clayey sand, Clayey sand with gravel
FINE-GRAINED SOILS 50% or more passing No. 200 sieve	SILTS AND CLAYS Liquid Limit less than 50%		ML	Silt, Sandy silt with gravel
			CL	Lean clay, Sandy lean clay with gravel
			OL	Organic clay, Sandy organic clay with gravel
	SILTS AND CLAYS Liquid Limit more than 50%		MH	Elastic silt, Sandy elastic silt with gravel
			CH	Fat clay, Sandy fat clay with gravel
			OH	Organic clay, Sandy organic clay with gravel
HIGHLY ORGANIC			PT	Peat, Highly organic silts

NOTE:
1. Coarse-grained soils receive dual symbols if: (a) their fines are CL-ML (e.g. SC-SM or GC-GM) or (b) they contain 5-12% fines (e.g. SW-SM, GP-GC, etc.). Fine-grained soils receive dual symbols if their limits plot in the hatched zone of the Plasticity Chart (CL-ML).
2. The table lists 30 out of a possible 110 Group Names, all of which are assigned to unique proportions of the constituent soils. Flow charts in ASTM D 2487-93 aid assignment of the Group Names.

PLASTICITY CHART



GRAIN SIZE CLASSIFICATION

CLASSIFICATION	US STANDARD SIEVE SIZE
BOULDERS	Above 12"
COBBLES	12" to 3"
GRAVEL Coarse Fine	 3" to 3/4" 3/4" to No. 4
SAND Coarse Medium Fine	 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No.200
SILT & CLAY	Below No. 200

COHESIVE SOIL CONSISTENCY

CLASSIFICATION	UNCONFINED COMP. STRENGTH (psf)
Very Soft	< 500
Soft	500 - 1000
Medium(Firm)	1000 - 2000
Stiff	2000 - 4000
Very Stiff	4000 - 8000
Hard	> 8000

COHESIONLESS SOIL RELATIVE DENSITY

CLASSIFICATION	SPT BLOW COUNTS (Blows/ft)
Very Loose	< 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	> 50

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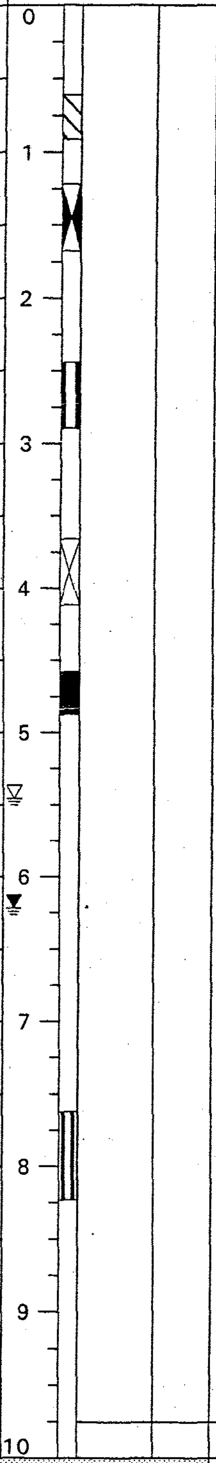
MATERIALS ENGINEERING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

JOB NO.: 99170.10
PLATE NO: A-1A

Boring Location, Elevation & Date Drilled: ; Elev. approx. m.; drilled on	Drilling Method: -cm dia.	**BORING NUMBER** **LEGEND**
	Sampling Method:	Sheet 1 of 1

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
1			22	
2	110	12	23	1.2
3	98	28	100 psi	0.8
4	-	10	35	-
5	95	20		0.7
6	-	-		12.0



Compressive strength as measured by Pocket Penetrometer, in tsf.	pp = 1.0tsf
2 inch I.D. California Sampler (C).	
2-1/2 inch I.D. Modified California Sampler (MC).	
3.5-inch I.D. Pitcher Tube Sampler (Cored).	
1-3/8 inch I.D. Standard Splitspoon Sampler (SPT).	
1.9 inch I.D. Hand Sampler driven with a slide hammer.	
Groundwater level first encountered during drilling	
Groundwater level at completion of boring	
Liquid Limit (LL), in percent Plasticity Index (PI), in percent	LL = 30 PI = 10
NX Core.	
Percent gravel and coarser in sample,(+#4) Percent fines (silt/clay) in sample, (-#200)	+#4 = 20% -#200 = 50%
Direct Shear Test	DS

LEGEND FOR LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000

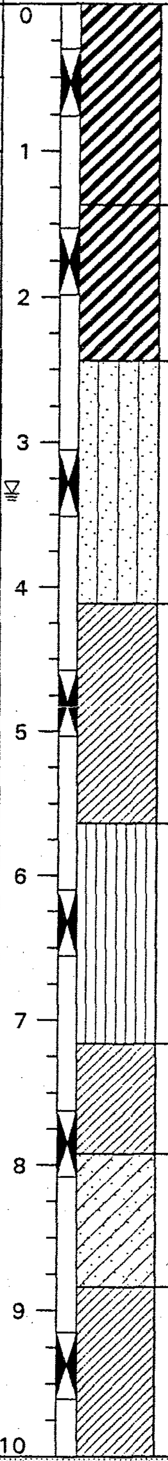
Job No.: 99106.10

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for omplete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may ffer at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:

A-1B

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 6.1 m.; drilled on 7-28-99	**Drilling Method:** 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-1**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 1 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	U.S.C.S.	Description	
MC-1	15.40	22	27		CH	Dark brown/gray with white pockets SANDY FAT CLAY, trace gravel, sand and gravel are angular to subangular, gravel up to 19 mm, pieces of glass, very stiff, moist (Fill)	
MC-2	14.93	24	47		CH	Dark gray to black with white pockets FAT CLAY, trace sand and gravel, rootlets, hard	pp = 350 kPa LL = 64 PI = 42
MC-3	19.17	15	54		SM	Brown to dark brown SILTY SAND, pockets of clay and fine rounded gravel, hard, moist to wet	
MC-4	15.56	26	20		CL	Light brown/gray, mottled LEAN CLAY, trace fine sand, stiff, wet	LL = 29 PI = 9
MC-5	16.66	21	28		ML	Gray/green SANDY SILT,very fine grained sand, very stiff, wet	+ #4 = 0% -#200 = 63% pp = 75 kPa
MC-6	20.11	15	27		CL	Gray/green LEAN CLAY WITH SAND, trace rounded gravel up to 25 mm, very stiff, wet	pp = 50 kPa
					SC	Light gray CLAYEY SAND, trace gravel, sand is coarse and angular, medium dense, wet	
MC-7	15.71	25	26	115	CL	Light grayish brown with white pockets LEAN CLAY, trace sand, very stiff, wet	pp = 150 kPa

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

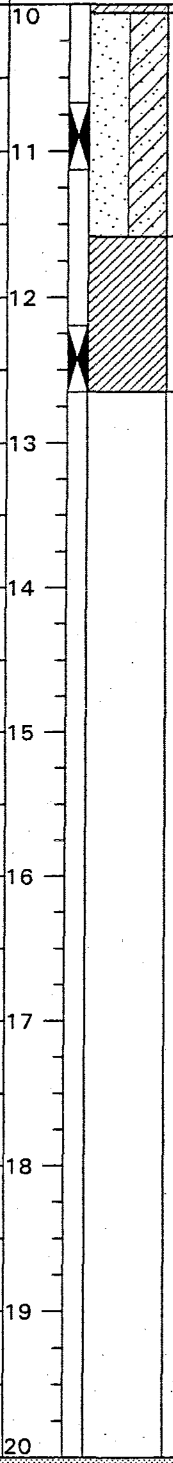
Date: 1/2000 Job No.: 99106.10

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-2A

MLB2 99106 1-3-00

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 6.1 m.; drilled on 7-28-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-56 rig	BORING NUMBER **B-1**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	U.S.C.S.	Description	
MC-8	18.54	16	35		SP SC	Brown POORLY GRADED SAND WITH CLAY, trace gravel, pockets of clay, medium dense, wet	+#4=9% -#200=9%
MC-9	15.87	24	26	96	CL	Light grayish brown with white pockets LEAN CLAY, very stiff, wet	pp=150 kPa
						Bottom of drillhole at 12.7 m Groundwater encountered at 3.4 m during drilling	

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000
Job No.: 99106.10

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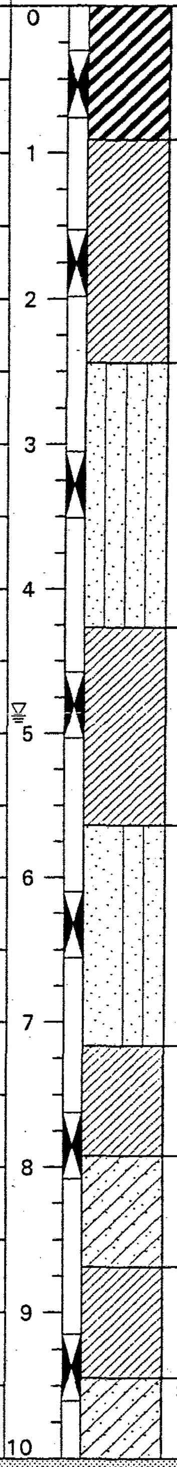
Plate:
A-2B

MLB2 99106 1-3-00

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 5.0 m.; drilled on 7-28-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-2**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-1	18.39	11	50/10 cm	512
MC-2	16.19	20	42	335
MC-3	16.03	23	24	
MC-4	16.19	25	21	57
MC-5	17.91	19	31	
MC-6	20.90	12	63	
MC-7	20.43	14	35	

Depth (m) Soil Graph & U.S.C.S.



U.S.C.S.	Description	
CH	Dark gray/brown with white coloration SANDY FAT CLAY, trace gravel, hard, damp (Fill)	
CL	Dark gray/black LEAN CLAY, trace sand and fine gravel, very stiff, damp to moist	LL=49 PI=31
SM	Light gray/brown mottled with orange SILTY SAND, pockets of clay, medium dense, moist	
CL	Light brown/gray mottled, LEAN CLAY, pockets of sand, stiff, wet	pp=50 kPa
SP SM	Brown POORLY GRADED SAND WITH SILT, trace gravel, pockets of clay, medium dense, wet	+#4=0% -#200=8%
CL	Brown/gray LEAN CLAY WITH SAND, sand is coarse and angular, very stiff, wet	
SC	Brown CLAYEY SAND, trace gravel, dense, wet	
CL	Brown SANDY LEAN CLAY, sand is coarse and angular, very stiff, wet	LL=35 PI=20
SC	Brown/gray CLAYEY SAND, trace gravel, sand is	

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000
Job No.: 99106.10

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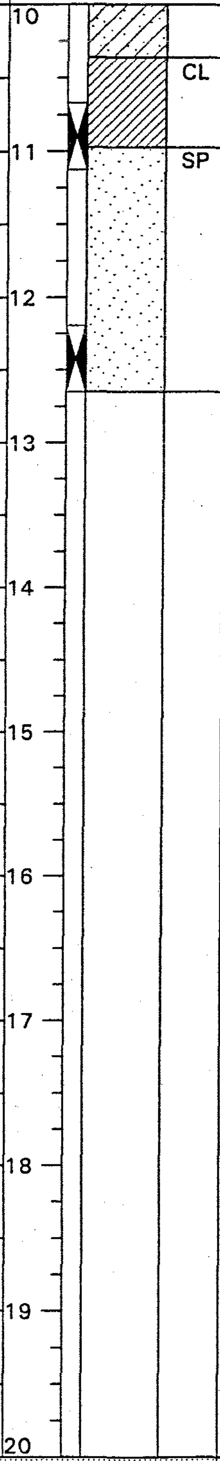
Plate:
A-3A

MLB2 99106 1-3-00

Boring Location, Elevation & Date Drilled:	Drilling Method:	BORING NUMBER
SEE ATTACHED SITE PLAN; Elev. approx. 5.0 m.; drilled on 7-28-99	20-cm dia. Hollow stem auger Mobil B-56 rig	B-2
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m) Soil Graph & U.S.C.S.	Description	
					10	coarse and angular, medium dense, wet	
MC-8	15.87	25	35		CL	Light brown/gray, mottled orange LEAN CLAY, very stiff, wet	pp = 50 kPa
					11 SP	Brown POORLY GRADED SAND, trace gravel, sand is coarse and subrounded to subangular, medium dense, wet	
MC-9	19.49	15	41		12		
					13	Bottom of drillhole at 12.7 m Groundwater encountered at 4.9 m during drilling	



LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000

Job No.: 99106.10

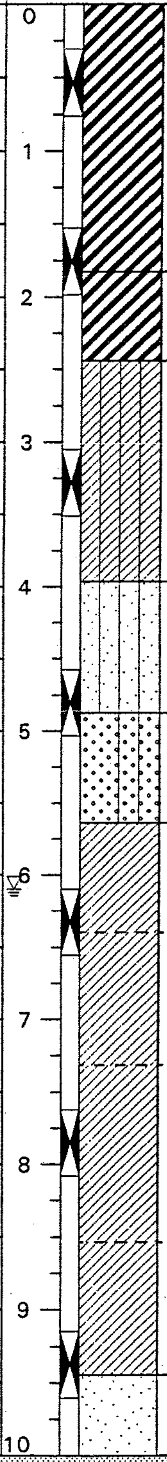
This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-3B

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 5.0 m.; drilled on 7-28-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-3**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-1	17.29	16	11	
MC-2	18.54	12	25	129
MC-3	19.33	14	15	
MC-4	20.11	11	50/15 cm	
MC-5	15.56	24	42	48
MC-6	15.24	28	22	
MC-7	16.34	21	49	

Depth (m) Soil Graph & U.S.C.S.



U.S.C.S.	Description	
CH	Dark brown to dark gray, white coloration, SANDY FAT CLAY, trace gravel, firm, dry to damp (Fill)	
CH	Dark brown to dark gray FAT CLAY WITH SAND, occasional gravel, very stiff, moist	
CL ML	Gray/light brown SILTY CLAY, stiff, moist	pp = 150 kPa LL = 23 PI = 7
SM	Grayish brown SILTY SAND, pockets of clay	
SW SM	Brown WELL GRADED SAND WITH SILT AND GRAVEL, gravel up to 19 mm, very dense, wet	+#4 = 33% -#200 = 5%
CL	Brown/gray, mottled orange SANDY LEAN CLAY, very fine sand, very stiff, wet	
CL	Brown/gray LEAN CLAY WITH SAND, very fine sand, very stiff, wet	pp = 125 kPa
CL	Grayish green LEAN CLAY, trace fine sand, occasional gravel, stiff, wet	pp = 300 kPa
CL	Grayish brown LEAN CLAY WITH SAND, sand is coarse and angular, trace angular gravel, very stiff, wet	
SP	Gray POORLY GRADED SAND, trace gravel, pockets of	

LOG OF BORING

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PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000 Job No.: 99106.10

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Plate: A-4A

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 5.0 m.; drilled on 7-28-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-3**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m)	Soil Graph & U.S.C.S.		
					10		clay, sand is coarse grained, subrounded to subangular, dense, wet	
MC-8	15.24	29	18		11	CL	Gray/brown, mottled orange/rust brown LEAN CLAY, trace very fine sand, stiff, wet	pp = 75 kPa
MC-9	15.56	27	17		12	CL	Brown LEAN CLAY, stiff, wet	pp = 75 kPa LL = 36 PI = 19
					13		Bottom of drillhole at 12.7 m Groundwater encountered during drilling at 6.1 m	
					14			
					15			
					16			
					17			
					18			
					19			
					20			

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000

Job No.: 99106.10

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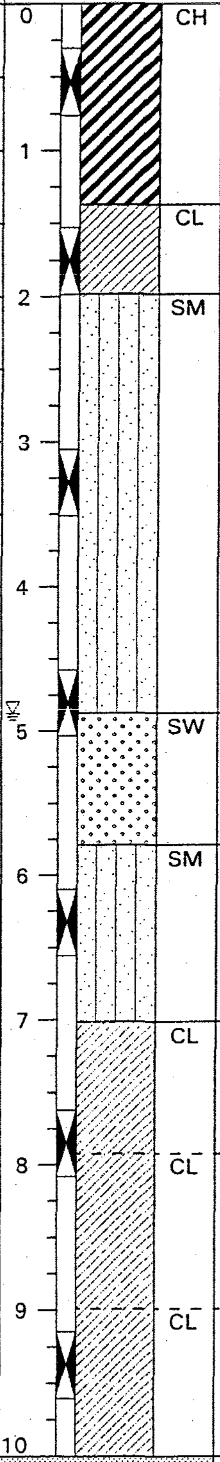
Plate:
A-4B

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 5.0 m.; drilled on 7-29-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-4**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 1 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m)	U.S.C.S.	Description	
					0	CH	Brown/dark gray FAT CLAY, trace sand and gravel, stiff, damp to moist (Fill)	
MC-1	17.44	18	16	215				pp = 225 kPa
					1			
MC-2	16.97	20	28	268		CL	Brown to dark brown SANDY LEAN CLAY, trace gravel, very stiff, moist (Fill)	pp = 225 kPa LL = 37 PI = 19
					2	SM	Light gray SILTY SAND, pockets of clay, medium dense, moist	
MC-3	17.76	14	22		3			
					4			
MC-4	17.76	18	19				Dark greenish brown	
					▽ 5	SW	Brown WELL GRADED SAND, trace gravel, sand is fine to coarse grained, gravel is subrounded to rounded. loose, wet	
					6	SM	Brown SILTY SAND, trace gravel, clay pockets, medium dense, moist to wet	
MC-5	18.54	17	22					
					7	CL	Grayish green LEAN CLAY WITH SAND, sand is coarse grained and angular, trace gravel, hard, wet	
MC-6	17.44	20	62	297				
					8	CL	Grayish green LEAN CLAY, trace scattered peat, hard, wet	pp = 275 kPa
					9	CL	Light brown, mottled rust brown LEAN CLAY, trace fine gravel, hard, wet	
MC-7	16.66	23	57	239				pp = 300 kPa LL = 39 PI = 22
					10			

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

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MOUNTAIN VIEW, CA

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Plate:
A-5A

Boring Location, Elevation & Date Drilled: ;EE ATTACHED SITE PLAN; Elev. approx. 5.0 m.; drilled on 7-29-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-4**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m) Soil Graph & U.S.C.S.		
					10		
					CL	Gray/brown, mottled rust brown LEAN CLAY, stiff, wet	pp = 75 kPa
MC-8	16.03	22	20	38	11		
					CL	Gray LEAN CLAY, hard, wet	pp = 25 kPa
MC-9	15.56	28	48		12		
					13	Bottom of drillhole at 12.7 m Groundwater encountered at 4.9 m during drilling	
					14		
					15		
					16		
					17		
					18		
					19		
					20		

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000

Job No.: 99106.10

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Plate:

A-5B

Boring Location, Elevation & Date Drilled:
SEE ATTACHED SITE PLAN; Elev. approx. 4.2 m.; drilled on 7-29-99

Drilling Method:
20-cm dia. Hollow stem auger
Mobil B-56 rig

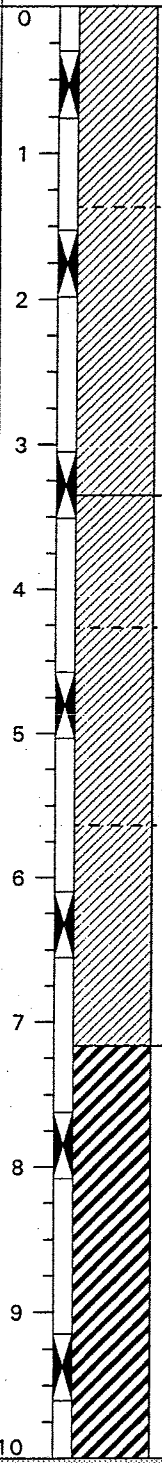
BORING NUMBER
B-5

Sampling Method:
64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.

Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-1	18.23	10	42	
MC-2	17.76	16	42	
MC-3	15.87	21	9	
MC-4	17.13	21	14	67
MC-5	15.56	28	17	
MC-6	16.19	24	22	
MC-7	16.66	22	33	239

Depth (m) Soil Graph & U.S.C.S.



U.S.C.S.	Description	
CL	Dark brown SANDY LEAN CLAY WITH GRAVEL, very stiff, dry, rootlets (Fill)	
CL	Light gray, LEAN CLAY WITH SAND, very stiff (Fill)	
	Dark brown, sand is fine to coarse and angular	
CL	Dark brown LEAN CLAY, trace sand, firm, damp to moist	pp = 250 kPa
CL	Light brown with white coloration, LEAN CLAY, trace sand with concretion, stiff, moist	pp = 75 kPa
CL	Gray/brown with rust mottling, LEAN CLAY, trace fine sand, stiff, moist	pp = 75 kPa
CH	Light brown with white coloration, FAT CLAY, stiff, moist	pp = 250 kPa
	Trace sand and fine gravel, very stiff, moist	pp = 350 kPa

LOG OF BORING

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MOUNTAIN VIEW, CA

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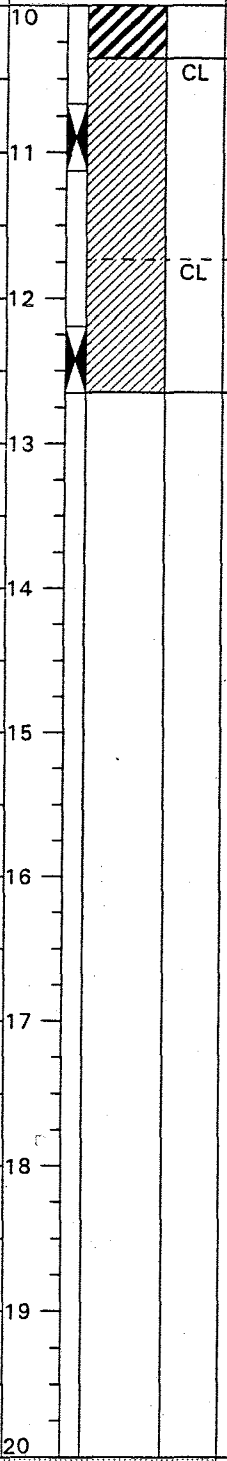
Plate:
A-6A

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 4.2 m.; drilled on 7-29-99	**Drilling Method:** 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-5**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m)	U.S.C.S.	Description	
					10			
MC-8	14.46	31	18	57		CL	Grayish brown with rust brown mottled, LEAN CLAY, trace sand, stiff, moist to wet	pp = 75 kPa
MC-9	17.60	21	36			CL	Gray/brown with rust brown mottled, LEAN CLAY WITH SAND, trace fine gravel, very stiff	pp = 200 kPa
							Bottom of drillhole at 12.7 m No groundwater encountered during drilling	

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000

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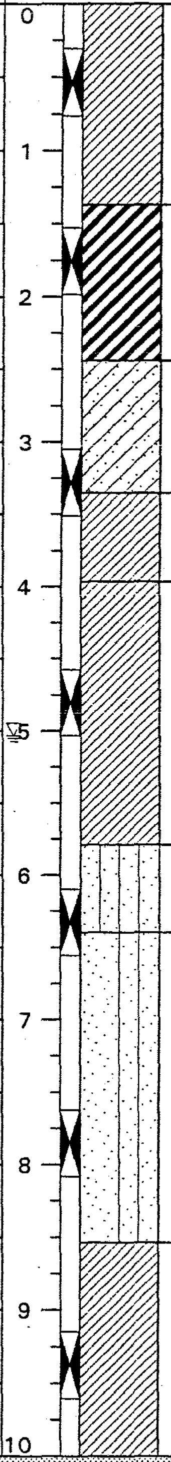
Plate:
A-6B

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 4.4 m.; drilled on 7-29-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-6**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-1	17.76	10	68	
MC-2	16.97	20	30	
MC-3	15.40	20	38	
MC-4	16.66	21	17	
MC-5	17.13	20	55	
MC-6	-	12	50/10 cm	
MC-7	15.87	24	41	

Depth (m) Soil Graph & U.S.C.S.



U.S.C.S.	Description	
CL	Light brownish/gray LEAN CLAY, trace sand and gravel, hard, damp (Fill)	
CH	Dark brown/olive green FAT CLAY, pieces of concrete up to 13 cm, very stiff, damp (Fill)	
SC	Dark grayish brown CLAYEY SAND WITH GRAVEL, medium dense, damp (Fill)	
CL	Dark gray/black LEAN CLAY, very stiff, damp (Fill)	
CL	Light gray/brown LEAN CLAY, trace sand, stiff, moist	pp = 100 kPa
SM	Gray SILTY SAND, medium dense, wet	
SP SM	Gray to dark brown POORLY GRADED SAND WITH SILT, sand is medium grained, medium dense, wet Trace gravel, very dense	
CL	Light brown with rust mottling LEAN CLAY, very stiff, wet	pp = 300 kPa LL = 45 PI = 27

LOG OF BORING

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MOUNTAIN VIEW, CA

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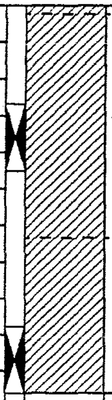
Plate:
A-7A

MLB2 99106 1-3-00

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 4.4 m.; drilled on 7-29-99	**Drilling Method:** 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-6**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m)	U.S.C.S.	Description	
					10	CL	Brown/gray SANDY LEAN CLAY, firm, wet	
MC-8	-	31	13					
					11			
						CL	Yellowish brown with rust mottling LEAN CLAY, some medium grained sand, very stiff, wet	
					12			
MC-9	16.34	24	31					pp = 150 kPa
							Bottom of drillhole at 12.7 m Groundwater encountered at 5.0 m during drilling	
					13			
					14			
					15			
					16			
					17			
					18			
					19			
					20			

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

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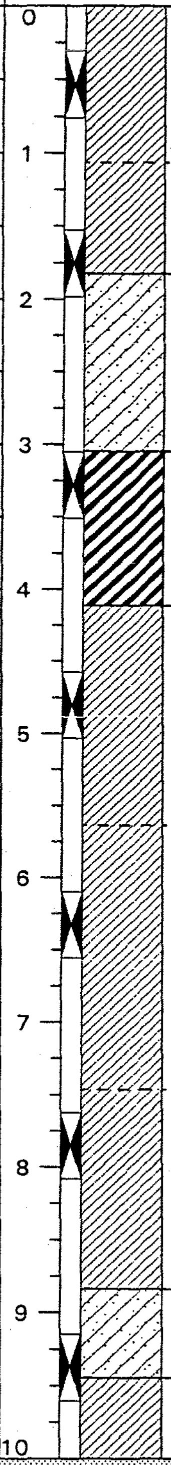
Plate:
A-7B

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 4.6 m.; drilled on 8-5-99	**Drilling Method:** 20-cm dia. Hollow stem auger Mobil B-40 rig	**BORING NUMBER** **B-7**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-1	16.81	10	68	
MC-2	20.27	8	44	
MC-3	15.09	29	38	
MC-4	16.34	29	17	
MC-5	16.03	24	29	
MC-6	16.19	26	18	
MC-7	17.13	22	40	

Depth (m) Soil Graph & U.S.C.S.



U.S.C.S.	Description	
CL	Light browm/gray LEAN CLAY WITH SAND, trace gravel, sand is very fine grained, hard, dry (Fill)	
CL	Greenish gray mottled with light brown, SANDY LEAN CLAY, very stiff, dry (Fill)	
SC	Dark brown CLAYEY SAND WITH GRAVEL, medium dense, dry (Fill)	
CH	Dark gray FAT CLAY, trace organics, very stiff, damp to moist (Fill)	pp = 175 kPa
CL	Brown/dark gray SANDY LEAN CLAY, trace gravel, stiff, moist	pp = 250 kPa
CL	Light gray, mottled rust brown LEAN CLAY, trace coarse sand, very stiff, moist	pp = 225 kPa
CL	Light brown/gray, mottled rust LEAN CLAY, trace sand and gravel, stiff, moist	pp = 75 kPa
SC	Brown CLAYEY SAND, coarse grained, medium dense, wet	
CL	Brown gray, rust brown mottled LEAN CLAY, very stiff, wet	pp = 350 kPa

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000 | Job No.: 99106.10

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Plate: A-8A

MLB2 99106 1-3-00

Boring Location, Elevation & Date Drilled:
EE ATTACHED SITE PLAN; Elev. approx. 4.6 m.; drilled on 8-5-99

Drilling Method:
20-cm dia. Hollow stem auger
Mobil B-40 rig

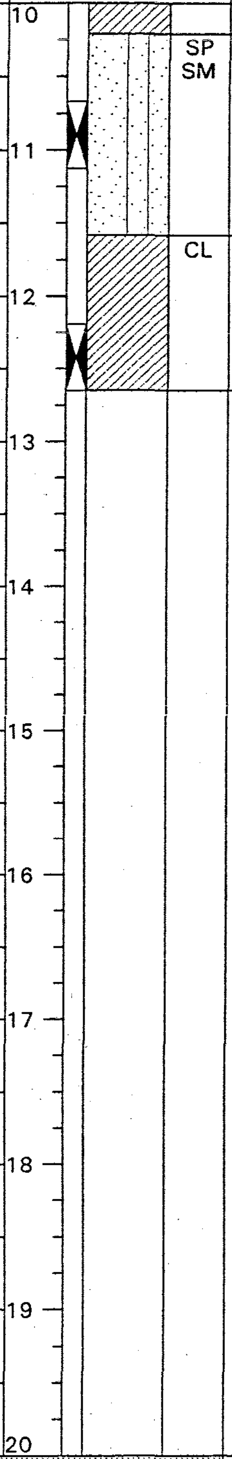
BORING NUMBER
B-7

Sampling Method:
64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.

Sheet 2 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-8	17.60	17	50/12.5 cm	
MC-9	15.24	27	25	

Depth (m) Soil Graph & U.S.C.S.



U.S.C.S.	Description	
SP SM	Gray POORLY GRADED SAND WITH SILT, very dense, wet	
CL	Greenish gray mottled with light brown LEAN CLAY, very stiff	pp = 75 kPa
	Bottom of drillhole at 12.7 m No groundwater encountered during drilling	

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000

Job No.: 99106.10

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Plate:
A-8B

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 2.7 m.; drilled on 7-29-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-56 rig	**BORING NUMBER** **B-8**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m) Soil Graph & U.S.C.S.			
					0	CL	Dark gray/brown SANDY LEAN CLAY WITH GRAVEL, gravel up to 25 mm, subangular, hard, dry to damp (Fill)	
MC-1	18.86	13	56					
					1	CH	Dark gray/brown FAT CLAY, trace sand and gravel, very stiff, damp (Fill)	
MC-2	16.66	20	43					pp = 350 kPa
					2	CL	Grayish brown SANDY LEAN CLAY, trace gravel, very stiff, damp (Fill)	
						CH	Dark gray FAT CLAY, very stiff, damp	pp = 400 kPa
					3			
MC-3	18.23	14	38					
					4	CL	Light brown/gray, mottled with brown LEAN CLAY, trace sand and gravel, stif, moist to wet	
MC-4	16.97	21	18	81				
					▽5			pp = 125 kPa
					6	SP SM	Gray POORLY GRADED SAND WITH SILT, medium to coarse grained sand, pockets of clay, dense, wet	
MC-5	17.13	15	50/15 cm					
					7	SM	Dark gray SILTY SAND, sand is coarse grained, trace fine gravel, medium dense, wet	+#4 = 8% -#200 = 34%
MC-6	17.60	20	47					
					8	CL	Light brown SANDY LEAN CLAY, very stiff, moist	pp = 100 kPa
					9			
MC-7	18.07	16	65			SP SC	Yellowish brown POORLY GRADED SAND WITH CLAY, sand is medium grained, trace gravel, dense, wet	
					10			

LOG OF BORING

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

Date: 1/2000

Job No.: 99106.10

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Plate:
A-9A

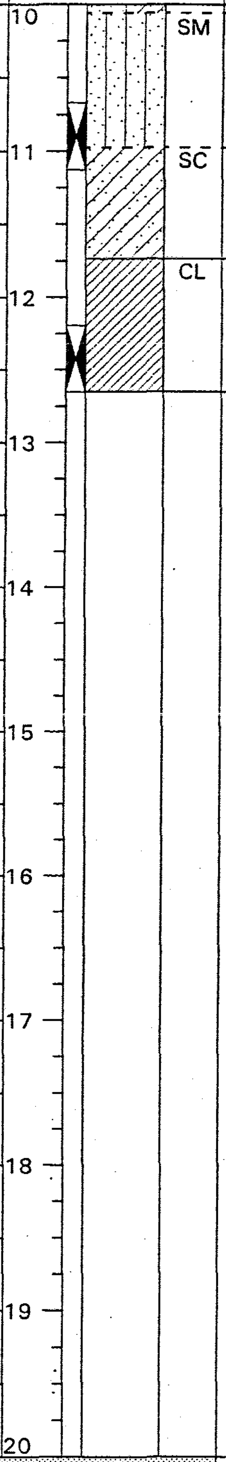
Boring Location, Elevation & Date Drilled:
SEE ATTACHED SITE PLAN; Elev. approx. 2.7 m.; drilled on 7-29-99

Drilling Method:
20-cm dia. Hollow stem auger
Mobil B-56 rig

BORING NUMBER
B-8

Sampling Method:
64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.

Sheet 2 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m)	U.S.C.S.	Description	
					10	SM	Light brown SILTY SAND, pockets of clay, medium dense, wet	
MC-8	16.19	25	25	34				
					11	SC	Brown CLAYEY SAND, sand is fine grained, medium dense, wet	
					12	CL	Yellowish brown/gray, mottled rust LEAN CLAY, very stiff, moist	
MC-9	14.77	28	36					pp = 300 kPa
					13		Bottom of drillhole at 12.7 m Groundwater encountered at 5.0 m during drilling	
					14			
					15			
					16			
					17			
					18			
					19			
					20			

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000

Job No.: 99106.10

Plate:
A-9B

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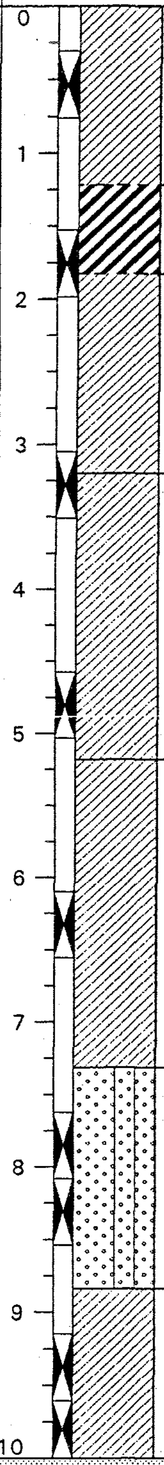
Boring Location, Elevation & Date Drilled: ;EE ATTACHED SITE PLAN; Elev. approx. 2.5 m.; drilled on 8-5-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-40 rig	**BORING NUMBER** **B-9**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-1	18.54	14	48	551
MC-2	18.07	18	15	
MC-3	17.76	18	34	
MC-4	15.40	14	34	43
MC-5	16.97	22	28	105
MC-6			33	
MC-7	18.54	11	44	
MC-8			49	
MC-9	16.50	24	53	

Depth (m) Soil Graph & U.S.C.S.



U.S.C.S.	Description	
CL	Brown to grayish brown LEAN CLAY, trace sand and gravel, hard, dry to damp (Fill)	
CH	Brown/gray SANDY FAT CLAY, stiff, dry to damp (Fill)	pp = 200 kPa
CL	Yellowish brown/gray, mottled rust brown LEAN CLAY, trace coarse sand, stiff (Fill)	
CL	Dark gray to yellowish brown SANDY LEAN CLAY, very stiff, damp Trace gravel, very stiff	pp = 325 kPa
CL	Light gray/brown LEAN CLAY WITH SAND, trace gravel, very stiff, moist	pp = 125 kPa
SW SM	(no recovery) Gray WELL GRADED SAND WITH SILT, trace gravel, pockets of clay, medium dense	+ #4 = 0% -#200 = 5%
CL	(no recovery) Gray/brown LEAN CLAY, hard, moist to wet	

LOG OF BORING

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000
Job No.: 99106.10

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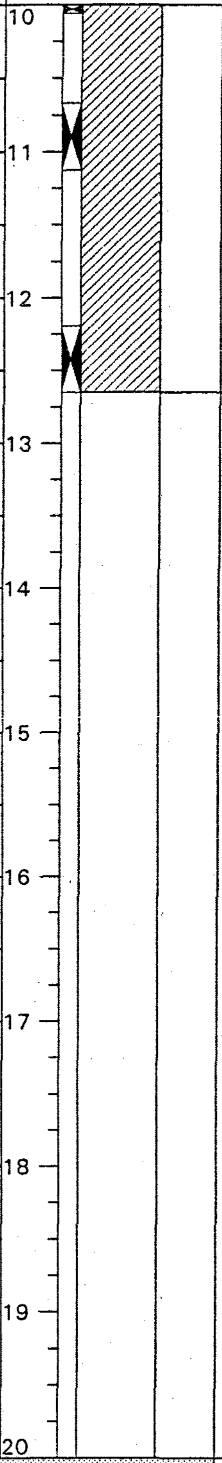
Plate:
A-10A

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 2.5 m.; drilled on 8-5-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-40 rig	**BORING NUMBER** **B-9**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2

ample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-10	15.87	25	48	230
MC-11	16.03	27	31	



Yellowish brown

Trace fine gravel, very stiff

Bottom of drillhole at 12.7 m
No groundwater encountered during drilling

pp = 175 kPa

pp = 275 kPa

pp = 75 kPa

LOG OF BORING

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Plate: A-10B

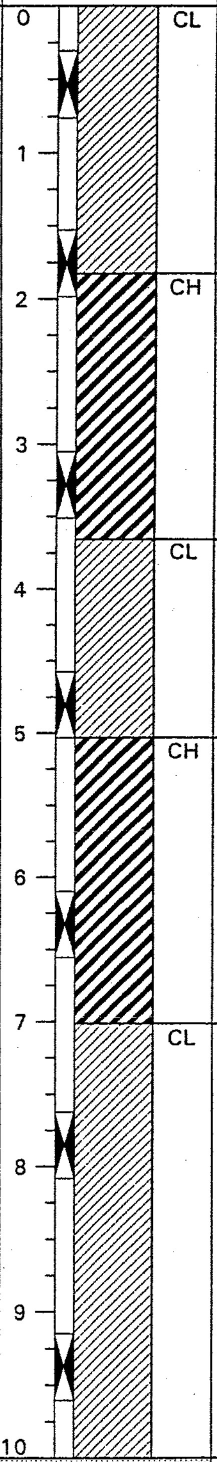
Boring Location, Elevation & Date Drilled:
SEE ATTACHED SITE PLAN; Elev. approx. 4.5 m.; drilled on 8-5-99

Drilling Method:
20-cm dia. Hollow stem auger
Mobil B-40 rig

Sampling Method:
64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.

BORING NUMBER
B-10

Sheet 1 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	U.S.C.S.	Description	
MC-1	19.17	10	35	479	CL	Light brown LEAN CLAY WITH SAND, trace fine gravel up to 13 mm, sand is fine to coarse, roots, very stiff, dry to damp (Fill)	
MC-2	16.50	18	37				
					CH	Dark gray to black FAT CLAY, very stiff, damp (Fill)	
MC-3	15.56	24	49			Gray to grayish brown, trace subrounded gravel, hard	pp = 350 kPa
					CL	Grayish brown SANDY LEAN CLAY, trace roots, stiff, moist (Fill)	
MC-4	15.09	28	15				pp = 100 kPa LL = 32 PI = 15
					CH	Gray FAT CLAY, stiff, moist	
MC-5	12.57	41	15				pp = 50 kPa
					CL	Gray/yellowish brown, mottled rust LEAN CLAY, very stiff, moist	
MC-6	15.24	26	44	144			pp = 350 kPa
MC-7	19.33	16	52	77		Lense of coarse sand between 9.5 and 9.6 m, hard	pp = 250 kPa LL = 41 PI = 24

LOG OF BORING

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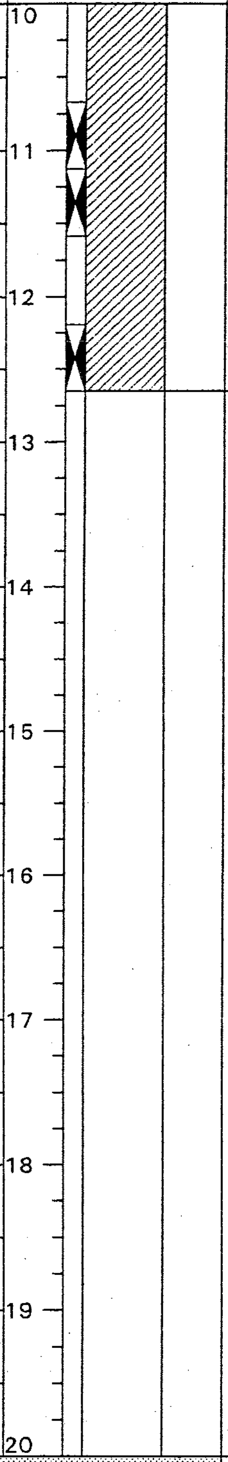
Plate:
A-11A

Boring Location, Elevation & Date Drilled: SEE ATTACHED SITE PLAN; Elev. approx. 4.5 m.; drilled on 8-5-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-40 rig	**BORING NUMBER** **B-10**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Description	
MC-8			37		(no recovery)	
MC-9	15.09	29	42		Trace sand, very stiff	pp = 50 kPa
MC-10	16.34	21	51	192	Hard	pp = 250 kPa
					Bottom of drillhole at 12.7 m No groundwater encountered during drilling	

LOG OF BORING

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Plate:
A-11B

Boring Location, Elevation & Date Drilled:
SEE ATTACHED SITE PLAN; Elev. approx. 4.2 m.; drilled on 9-22-99

Drilling Method:
20-cm dia. Hollow stem auger
Mobil B-40 rig

Sampling Method:
64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.

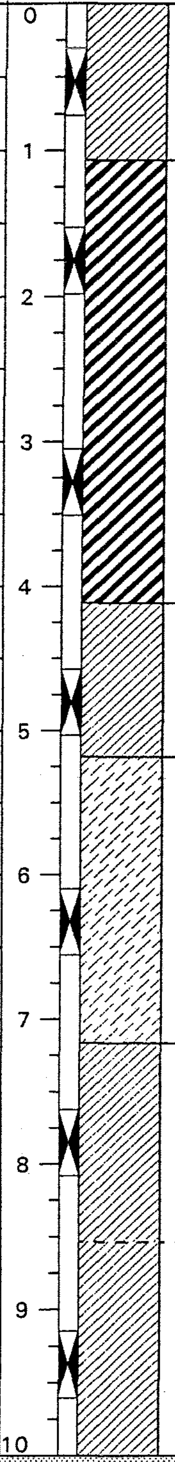
BORING NUMBER

B-11

Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)
MC-1	18.07	17	39	
MC-2	13.04	33	34	
MC-3	16.19	24	42	201
MC-4	16.19	25	31	
MC-5	8.96	68	14	67
MC-6	16.66	24	22	
MC-7	16.34	23	43	

Depth (m) Soil Graph & U.S.C.S.



U.S.C.S.	Description	
CL	Brown LEAN CLAY, occasional sand and gravel, very stiff, dry (Fill)	
CH	Dark gray FAT CLAY, pockets of lean clay, very stiff, moist (Fill)	pp = 175 kPa
	Dark gray, occasional gravel, very stiff, moist	pp = 300 kPa
CL	Dark gray, mottled with brown and light greenish gray LEAN CLAY, occasional fine gravel, very stiff, moist (Fill)	pp = 150 kPa
OH	Dark gray ORGANIC CLAY, stiff, moist	pp = 150 kPa LL = 119 PI = 81
CL	Light greenish gray SANDY LEAN CLAY, occasional gravel, stiff, moist	pp = 50 kPa
CL	Brown mottled with yellow and gray LEAN CLAY, occasional gravel, very stiff, moist Lense of clayey sand at 9.3 m, sand is coarse grained	pp = 350 kPa

LOG OF BORING

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Date: 1/2000

Job No.: 99106.10

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Plate:

A-12A

Boring Location, Elevation & Date Drilled: EE ATTACHED SITE PLAN; Elev. approx. 4.2 m.; drilled on 9-22-99	Drilling Method: 20-cm dia. Hollow stem auger Mobil B-40 rig	**BORING NUMBER** **B-11**
	Sampling Method: 64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m) Soil Graph & U.S.C.S.		
					10 ▨ CH	Brown FAT CLAY, hard, moist	
MC-8	15.40	27	69		11 ▨		pp = 350 kPa
					12 ▨		
MC-9	15.71	26	55		▨	Trace coarse sand, occasional gravel, hard	pp = 225 kPa
					13	Bottom of drillhole at 12.7 m No groundwater encountered during drilling	
					14		
					15		
					16		
					17		
					18		
					19		
					20		

LOG OF BORING

PARIKH CONSULTANTS, INC.
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PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

Date: 1/2000

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Plate:

A-12B

Boring Location, Elevation & Date Drilled:
SEE ATTACHED SITE PLAN; Elev. approx. 4.2 m.; drilled on 9-22-99

Drilling Method:
20-cm dia. Hollow stem auger
Mobil B-40 rig

Sampling Method:
64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.

BORING NUMBER
B-12

Sheet 1 of 2

Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m) Soil Graph & U.S.C.S.	Description	
MC-1	17.13	18	29		CL	Brown LEAN CLAY, trace sand and fine gravel, very stiff, damp (Fill)	
						Pocket of dark gray fat clay at 0.8 m	pp = 350 kPa
MC-2	17.91	15	38			Brown/greenish gray, trace gravel up to 38 mm, piece of sandstone up to 25 mm, very stiff	PP = 400 kPa LL = 30 PI = 12
MC-3	13.67	33	24	129	CH	Dark gray FAT CLAY, very stiff, moist (Fill)	pp = 270 kPa
MC-4	9.90	51	9	38		Dark gray/black, pockets of clayey sand, firm, moist	pp = 50 kPa
MC-5	8.33	73	10	48	OH	Dark gray ORGANIC CLAY, firm, moist	pp = 50 kPa LL = 109 PI = 72
MC-6	14.46	30	25		CH	Dark gray FAT CLAY, very stiff, moist Changes to light greenish gray at bottom of sample, trace sand	pp = 125 kPa
MC-7	13.51	31	25		CL	Light greenish gray SANDY LEAN CLAY, very stiff, moist	pp = 200 kPa

LOG OF BORING

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Plate:
A-13A

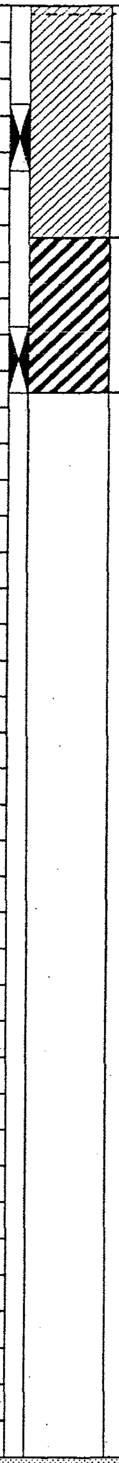
Boring Location, Elevation & Date Drilled:
SEE ATTACHED SITE PLAN; Elev. approx. 4.2 m.; drilled on 9-22-99

Drilling Method:
20-cm dia. Hollow stem auger
Mobil B-40 rig

BORING NUMBER
B-12

Sampling Method:
64 mm I.D. Mod. Calif. (MC), 64 kg (140 lbs) hammer, 76 cm (30 inch) drop.

Sheet 2 of 2



Sample Type & No.	Dry Density (kN/m3)	Water Content (%)	Blows Per 30 cm	Compress. Strength (kPa)	Depth (m)	Soil Graph & U.S.C.S.		
					10	CL	Brown mottled with gray LEAN CLAY, hard, moist	
MC-8	15.87	20	50					
					11			pp = 300 kPa
						CH	Brown/gray FAT CLAY, very stiff, moist	
					12			
MC-9	14.30	26	30					pp = 175 kPa
							Bottom of drillhole at 12.7 m No groundwater encountered during drilling	
					13			
					14			
					15			
					16			
					17			
					18			
					19			
					20			

LOG OF BORING

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Plate: A-13B

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System. The results are presented on "Log of Borings", Appendix A.

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-92. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-93. The results of these tests are presented on Plate B-2, "Plasticity Chart".

Grain Size Classification

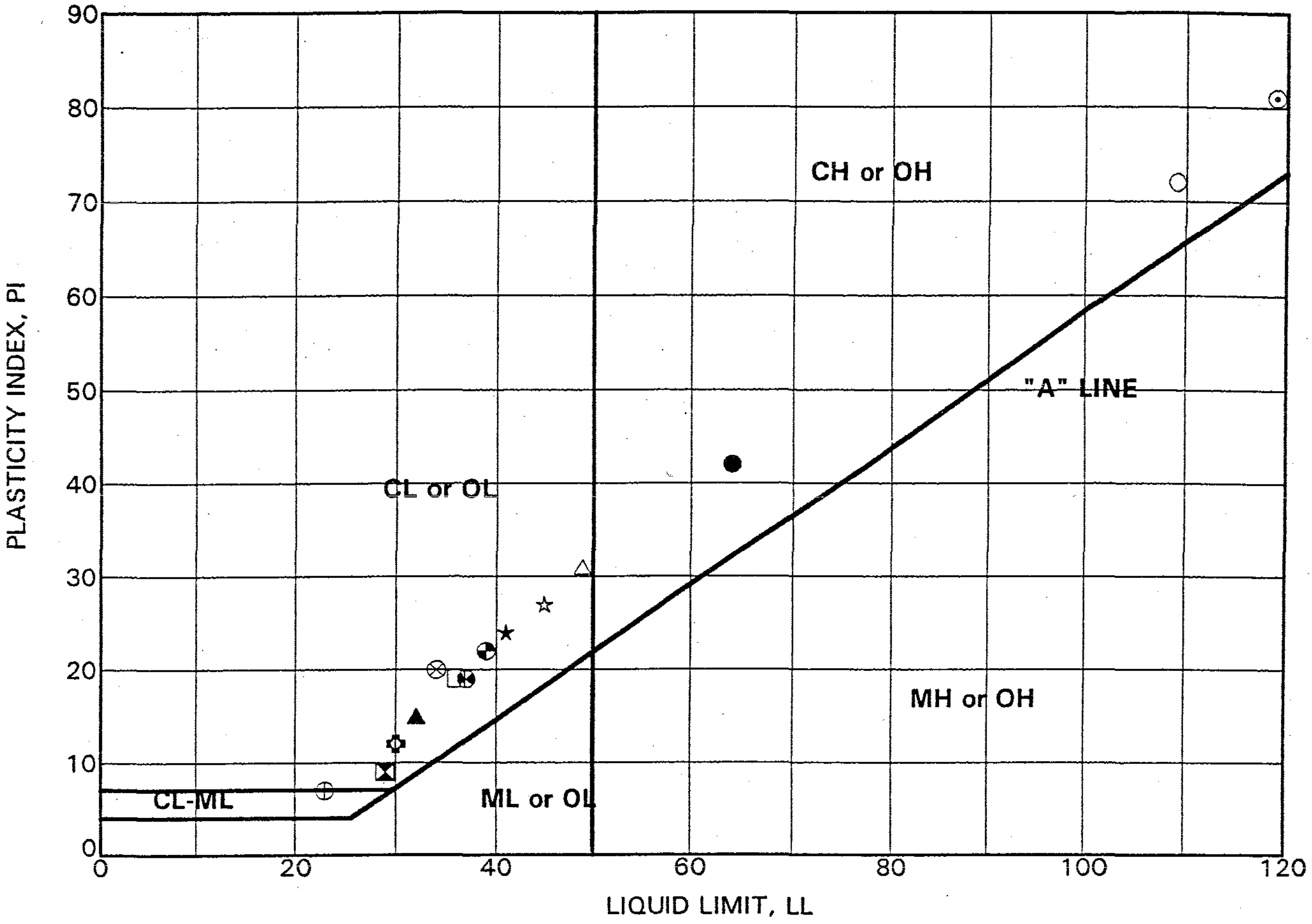
Grain size classification tests (ASTM Test Method D 422-63) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plates B-3A to B-3B, "Grain Size Distribution Curves".

Unconfined Compression Tests

Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression test was performed in general accordance with ASTM Test Method D 2166-91. The results are presented on "Log of Borings", Appendix A.

Triaxial Tests

Triaxial tests were performed on selected soil samples in general accordance with relevant ASTM standards. The tests were performed under consolidated undrained conditions and the pore water pressures generated during testing were measured. The tests results are presented on Plates B-4A to B-4J



PLASTICITY CHART

Boring Number	Sample Number	Depth (m)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
B-1	MC-2	1.7	●	24	64	22	42	FAT CLAY (CH)
B-1	MC-4	4.9	⧗	26	29	20	9	LEAN CLAY (CL)
B-10	MC-4	4.7	▲	28	32	17	15	LEAN CLAY (CL)
B-10	MC-7	9.4	★	16	41	17	24	LEAN CLAY (CL)
B-11	MC-5	6.2	⊙	68	119	38	81	ORGANIC CLAY (OH)
B-12	MC-2	1.7	✿	15	30	18	12	LEAN CLAY (CL)
B-12	MC-5	6.2	○	73	109	37	72	ORGANIC CLAY (OH)
B-2	MC-2	1.8	△	20	49	18	31	LEAN CLAY (CL)
B-2	MC-7	9.3	⊗	14	34	14	20	LEAN CLAY (CL)
B-3	MC-3	3.4	⊕	14	23	16	7	SILTY CLAY (CL-ML)
B-3	MC-9	12.3	□	27	36	17	19	LEAN CLAY (CL)
B-4	MC-2	1.7	◍	20	37	18	19	LEAN CLAY (CL)
B-4	MC-7	9.3	◕	23	39	17	22	LEAN CLAY (CL)
B-6	MC-7	9.3	☆	24	45	18	27	LEAN CLAY (CL)

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JOB NO: 99106.10
PLATE NO: B-2

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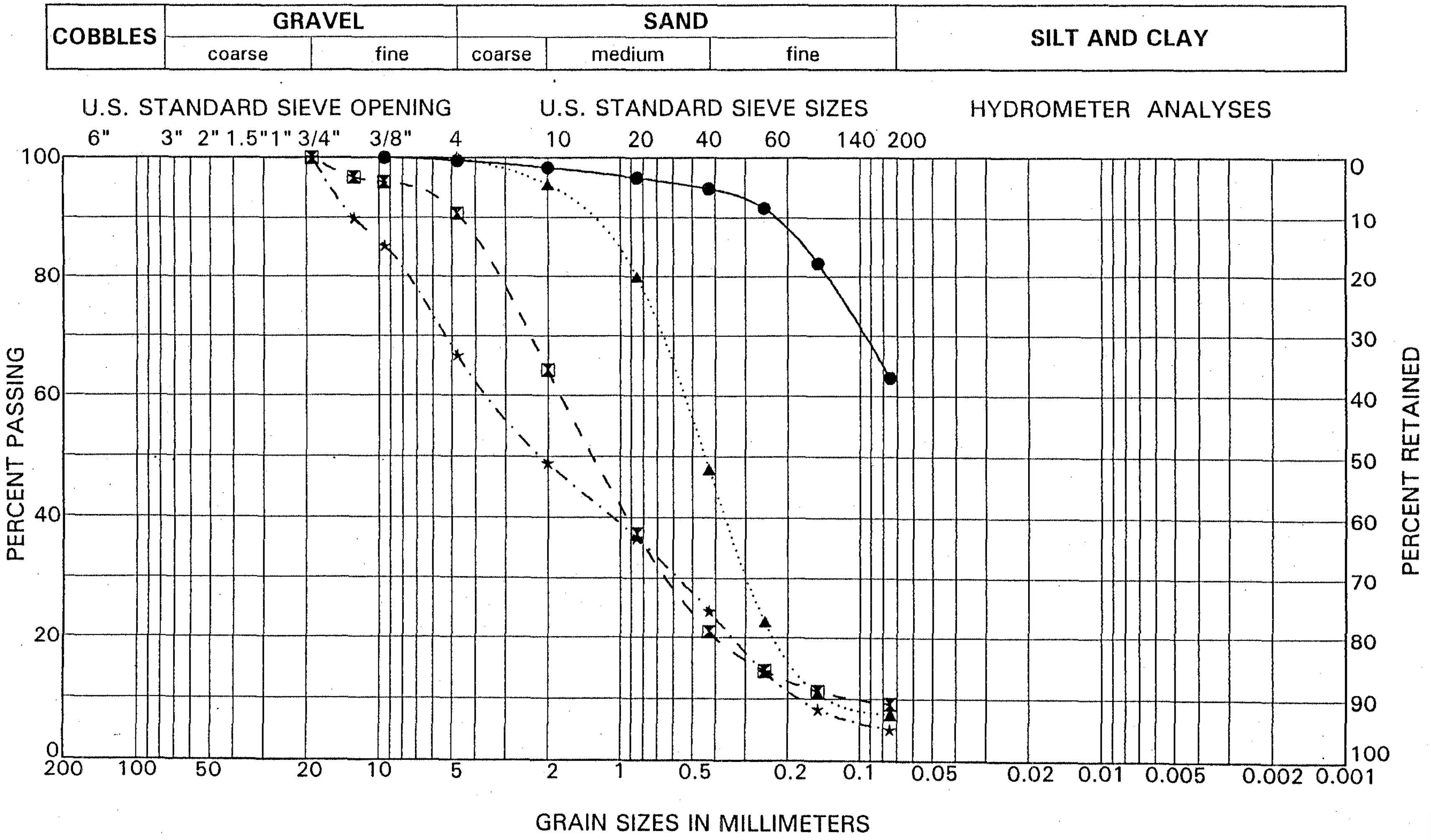
PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
MOUNTAIN VIEW, CA

JOB NO: 99106.10

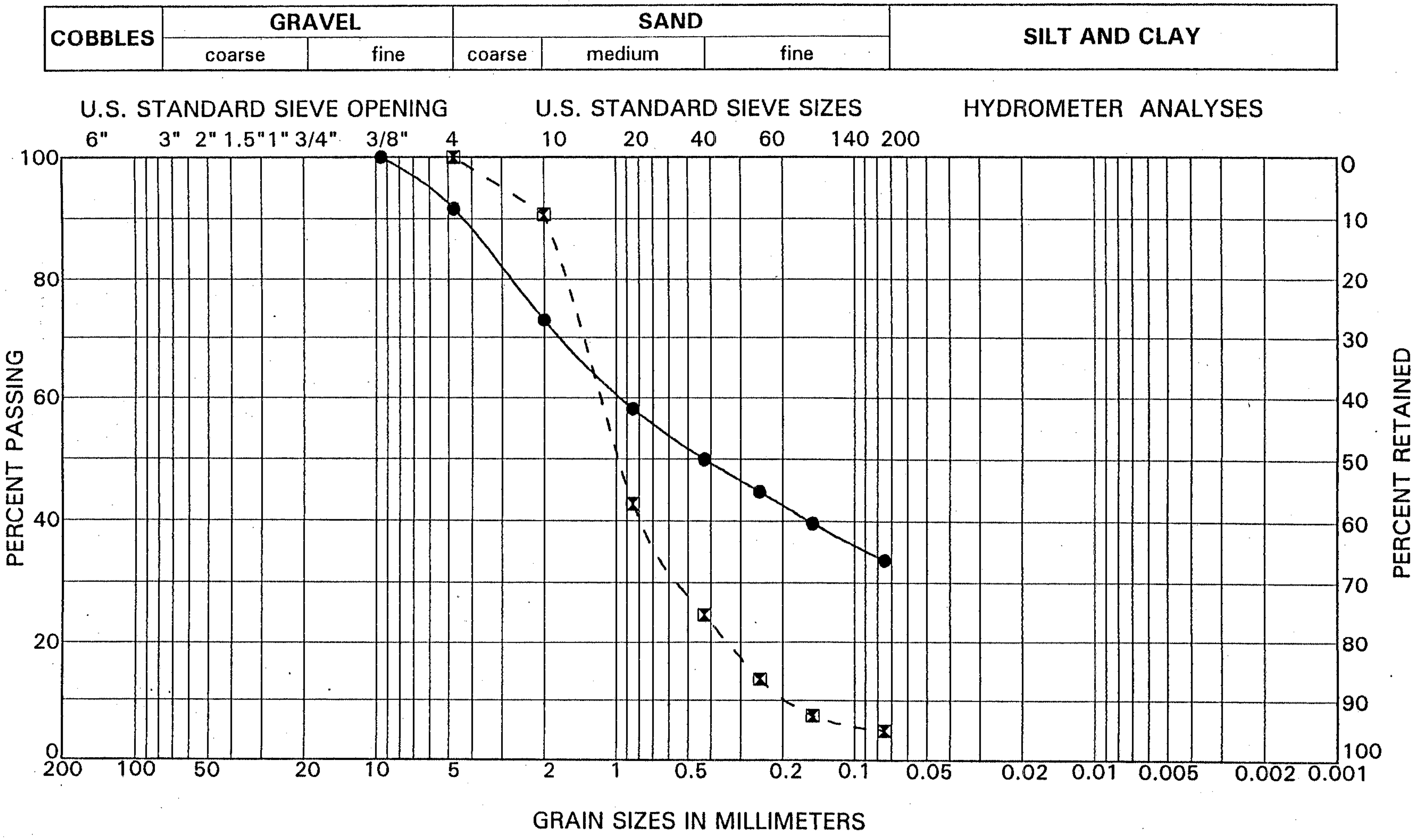
PLATE NO: B-3A

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (m)	Symbol	LL	PI	Description
B-1	MC-5	6.2	●			SANDY SILT (ML)
B-1	MC-8	11.0	☒			POORLY GRADED SAND WITH CLAY (SP-SC)
B-2	MC-5	6.4	▲			POORLY GRADED SAND WITH SILT (SP-SM)
B-3	MC-4	4.9	★			WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM)

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (m)	Symbol	LL	PI	Description
B-8	MC-6	7.8	●			SILTY SAND (SM)
B-9	MC-7	8.2	☒			WELL GRADED SAND WITH SILT (SW-SM)

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PERMANENTE CREEK LEVEES CERTIFICATION PROJECT
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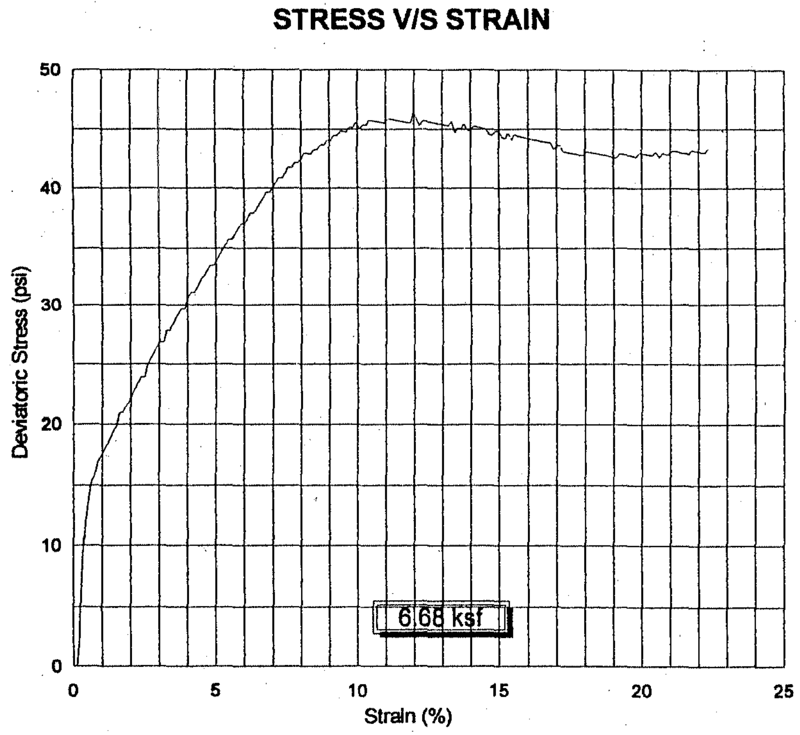
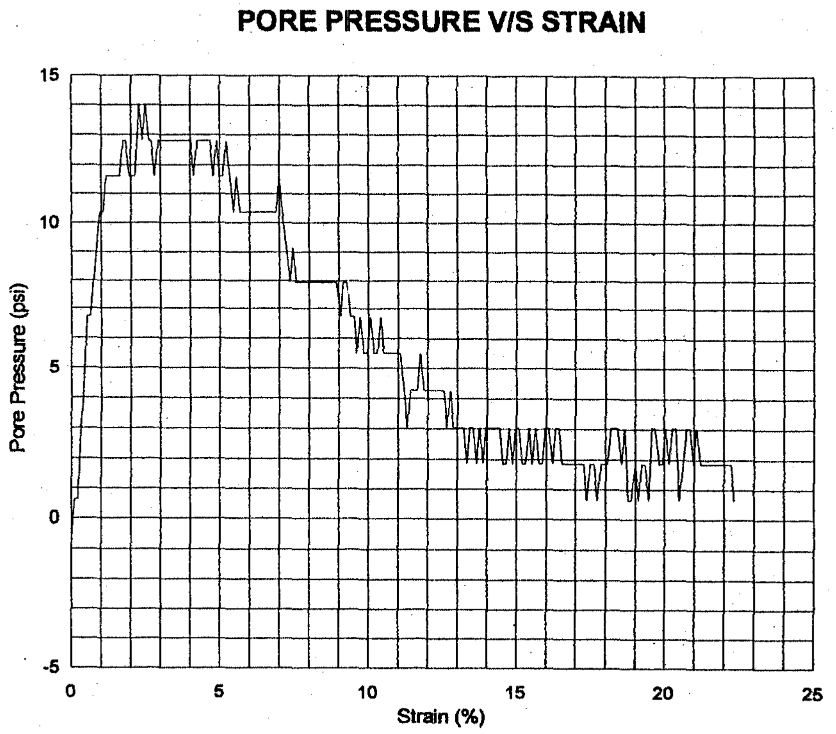
JOB NO: 99106.10

PLATE NO: B-3B

CONSOLIDATED UNDRAINED COMPRESSION TEST


STRESS V/S STRAIN


PORE PRESSURE V/S STRAIN

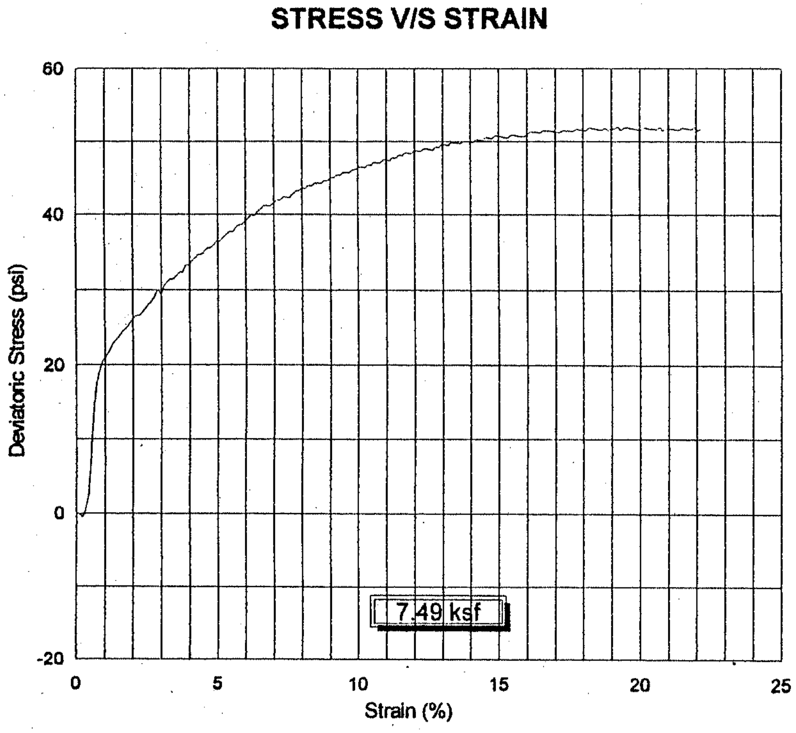
Height of Sample (inch)	5	Cell Pressure (psi)	20
Initial Diameter (inch)	2.416	Back Pressure (psi)	25
% Moisture - Before Test	- -	Maximum Strength (psi))	46.37
% Moisture - After Test	114.9	Strain @ Failure (%)	11.99
Dry Density (pcf)	114.9	Pore Pressure @Failure (psi)	4.28

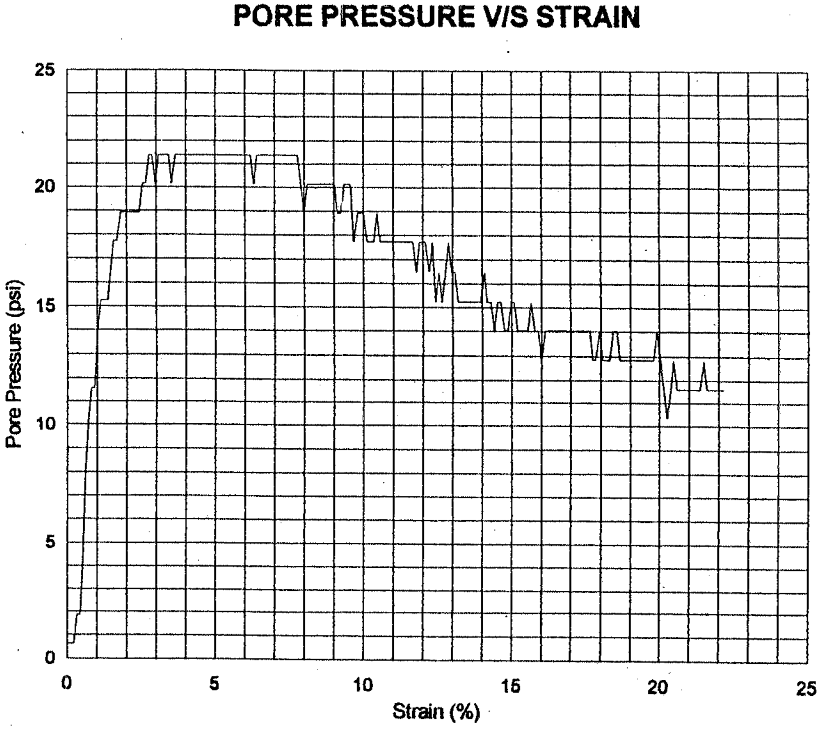
sigma3' (psi)= 15.72   sigma1' (psi)= 62.09

Boring No. B1   **Sample No.** 3B   **Elev. or Depth (.ft.)** 11

Job Name : Permanente Creek Study   **Job No.:** 99106.1   **Plate No.:** B-4A

CONSOLIDATED UNDRAINED COMPRESSION TEST





Height of Sample (inch)	5	Cell Pressure (psi)	30
Initial Diameter (inch)	2.416	Back Pressure (psi)	25
% Moisture - Before Test	26.9	Maximum Strength (psi))	52.02
% Moisture - After Test	22.4	Strain @ Failure (%)	19.22
Dry Density (pcf)	98.4	Pore Pressure @Failure (psi)	12.82

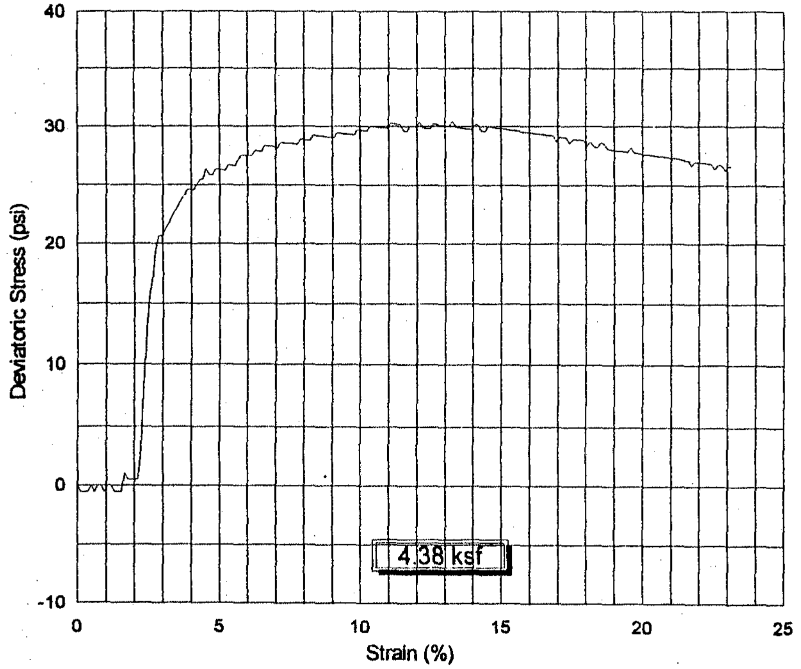
sigma3' (psi)= 17.18 sigma1' (psi)= 69.20

Boring No. B1 **Sample No.** 4B **Elev. or Depth (ft)** 16

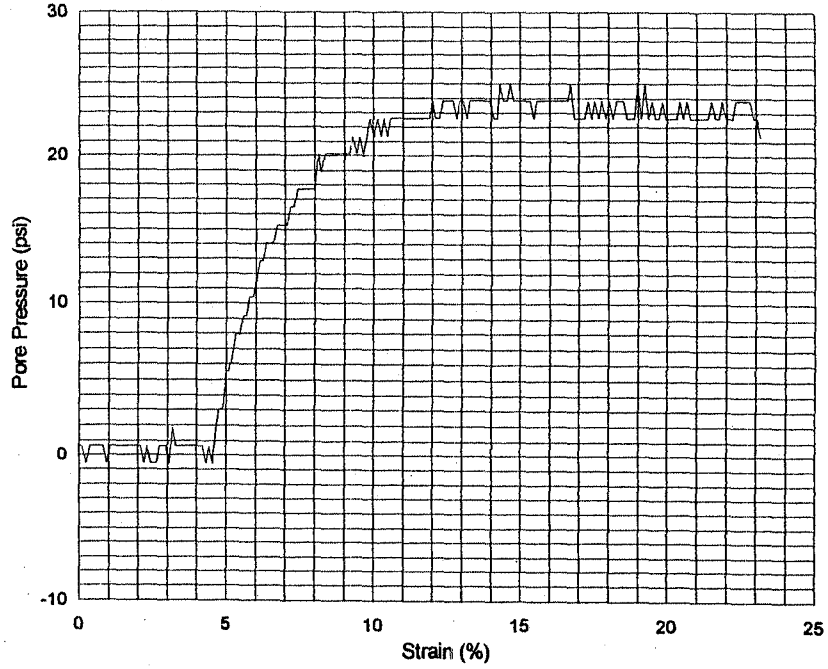
Job Name : Permanente Creek Study **Job No.:** 99106.1 **Plate No.:** B-4B

CONSOLIDATED UNDRAINED COMPRESSION TEST

STRESS V/S STRAIN



PORE PRESSURE V/S STRAIN

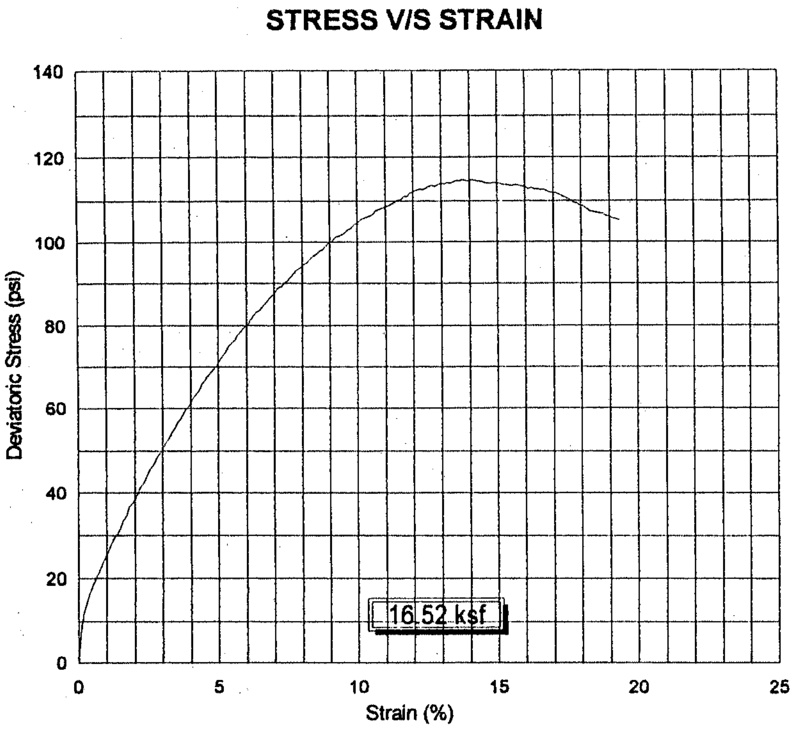


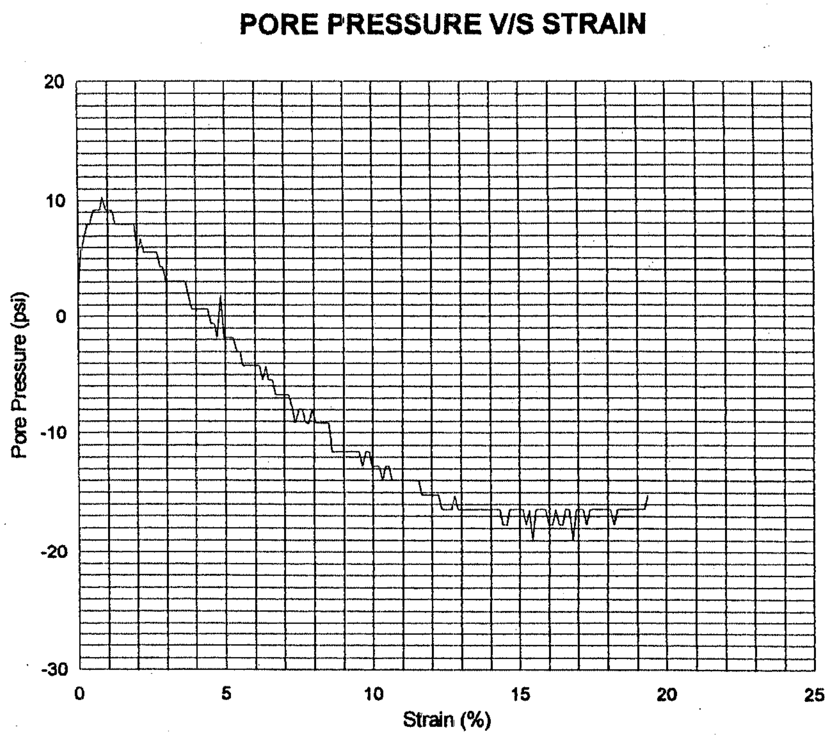
Height of Sample (inch)	5	Cell Pressure (psi)	35
Initial Diameter (inch)	2.416	Back Pressure (psi)	25
% Moisture - Before Test	29.2	Maximum Strength (psi))	30.45
% Moisture - After Test	22.2	Strain @ Failure (%)	13.28
Dry Density (pcf)	94.7	Pore Pressure @Failure (psi)	23.8
sigma3' (psi)=	11.2	sigma1' (psi)=	41.65

Boring No. B3 **Sample No.** 9A **Elev. or Depth (ft)** 40.5

Job Name : Premanente Creek **Job No.:** 99106.1 **Plate No:.** B-4C

CONSOLIDATED UNDRAINED COMPRESSION TEST





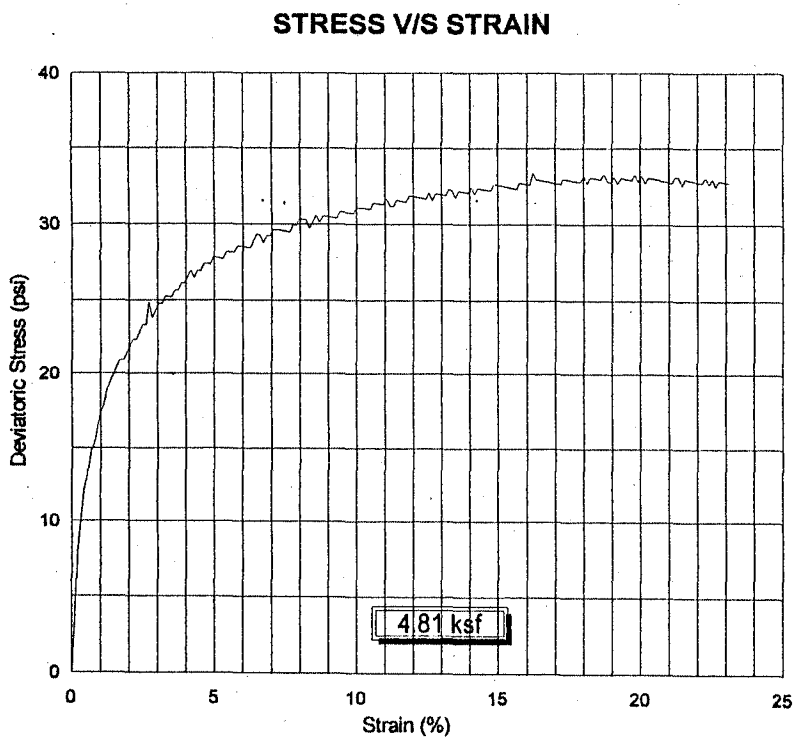
Height of Sample (inch)	5	Cell Pressure (psi)	20
Initial Diameter (inch)	2.416	Back Pressure (psi)	25
% Moisture - Before Test	18.2	Maximum Strength (psi))	114.75
% Moisture - After Test	18.8	Strain @ Failure (%)	14.10
Dry Density (pcf)	114.1	Pore Pressure @Failure (psi)	0

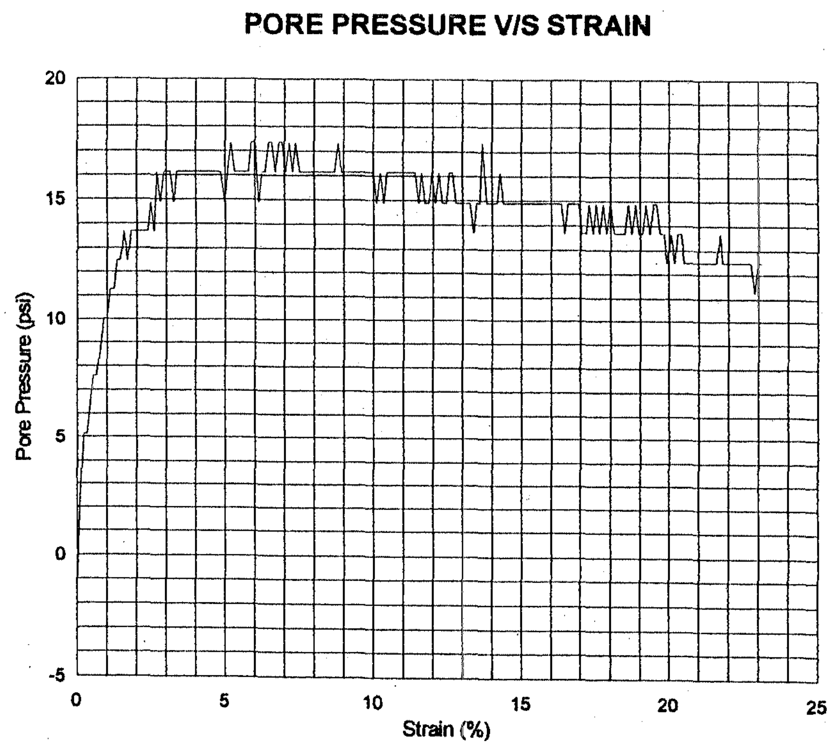
sigma3' (psi)= 36.46 sigma1' (psi)= 151.21

Boring No. B4 **Sample No.** 5A **Elev. or Depth (ft)** 20.5

Job Name : Permanente Creek Study **Job No.:** 99106.1 **Plate No.:** B-4D

CONSOLIDATED UNDRAINED COMPRESSION TEST


STRESS V/S STRAIN


PORE PRESSURE V/S STRAIN

Height of Sample (inch)	5	Cell Pressure (psi)	20
Initial Diameter (inch)	2.416	Back Pressure (psi)	40
% Moisture - Before Test	25.5	Maximum Strength (psi))	33.43
% Moisture - After Test	25.5	Strain @ Failure (%)	16.24
Dry Density (pcf)	99.1	Pore Pressure @Failure (psi)	14.9

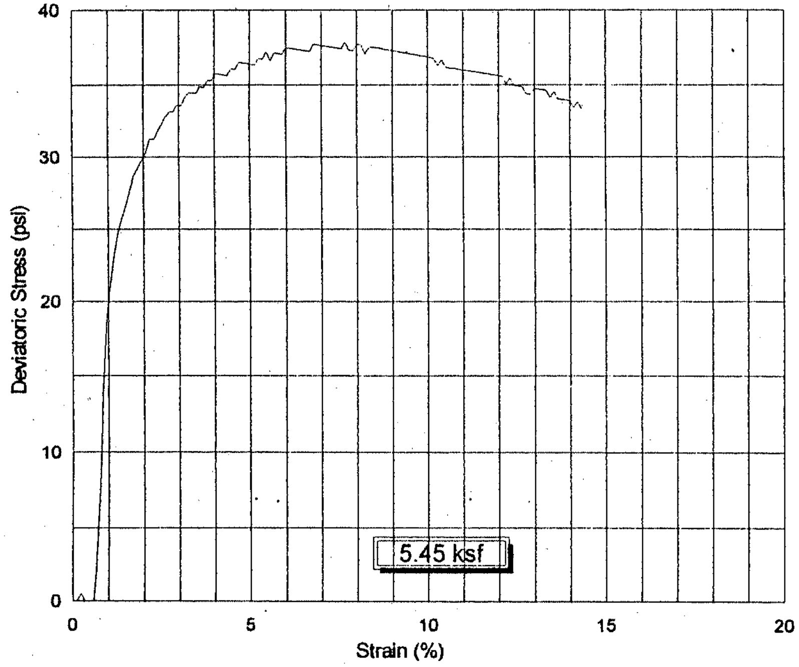
sigma3' (psi)= 5.1

sigma1' (psi)= 38.53

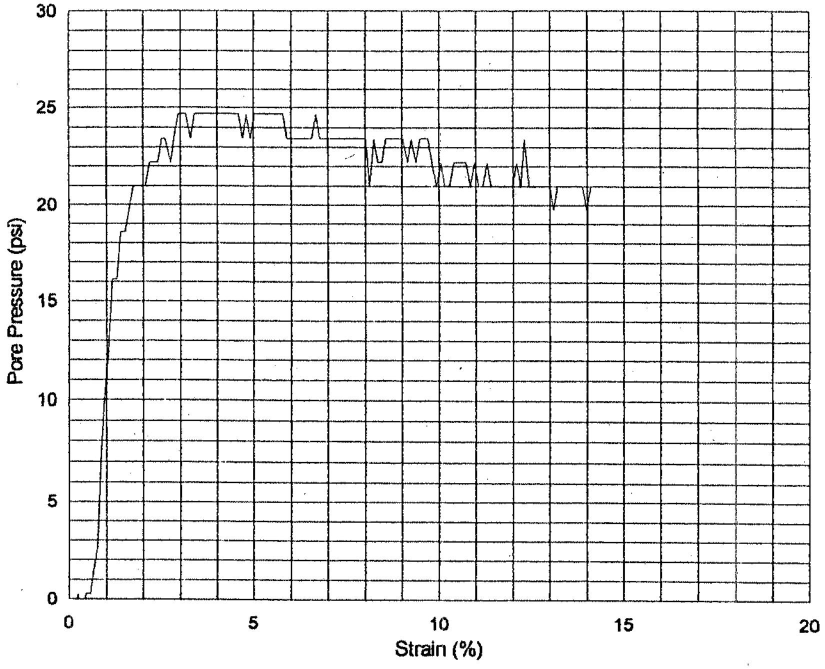
Boring No. B5 **Sample No.** 6A **Elev. or Depth (ft)** 25.5

Job Name : Premanente Creek **Job No.:** 99106.1 **Plate No:.** **B-4E**

CONSOLIDATED UNDRAINED COMPRESSION TEST



STRESS V/S STRAIN



PORE PRESSURE V/S STRAIN

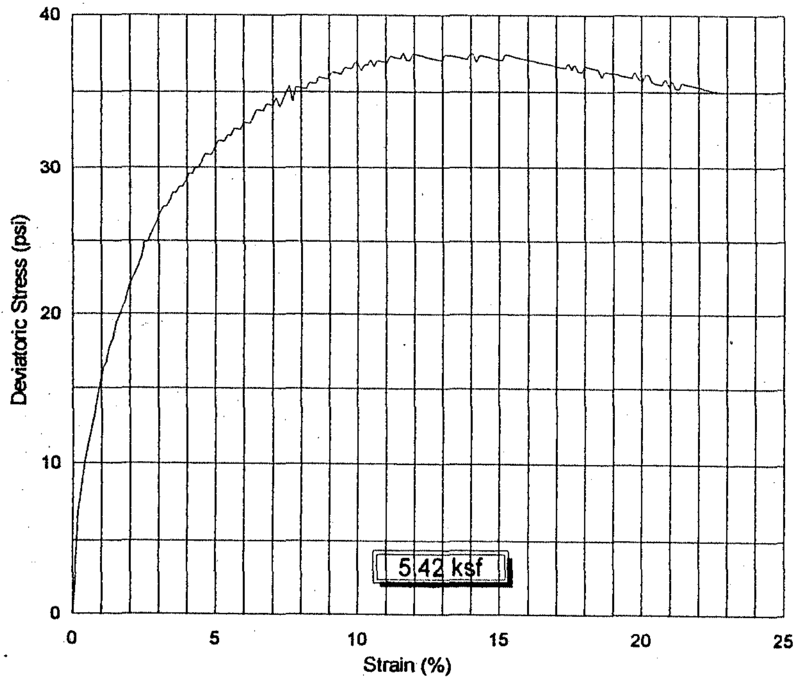
Height of Sample (inch)	5	Cell Pressure (psi)	35
Initial Diameter (inch)	2.416	Back Pressure (psi)	40
% Moisture - Before Test	28.5	Maximum Strength (psi))	37.83
% Moisture - After Test	- -	Strain @ Failure (%)	7.67
Dry Density (pcf)	94.2	Pore Pressure @Failure (psi)	23.44

sigma3' (psi)= 11.56 sigma1' (psi)= 49.39

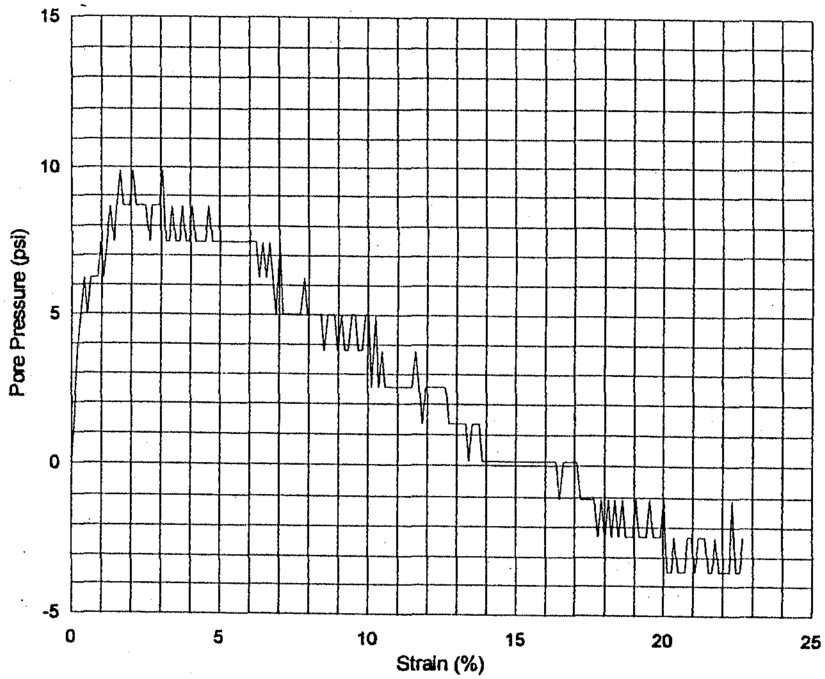
Boring No.	B5	**Sample No.**	8A	**Elev. or Depth (ft)**	35.5
Job Name :	Premanente Creek	**Job No.:**	99106.1	**Plate No:.**	B-4F

CONSOLIDATED UNDRAINED COMPRESSION TEST

STRESS V/S STRAIN



PORE PRESSURE V/S STRAIN

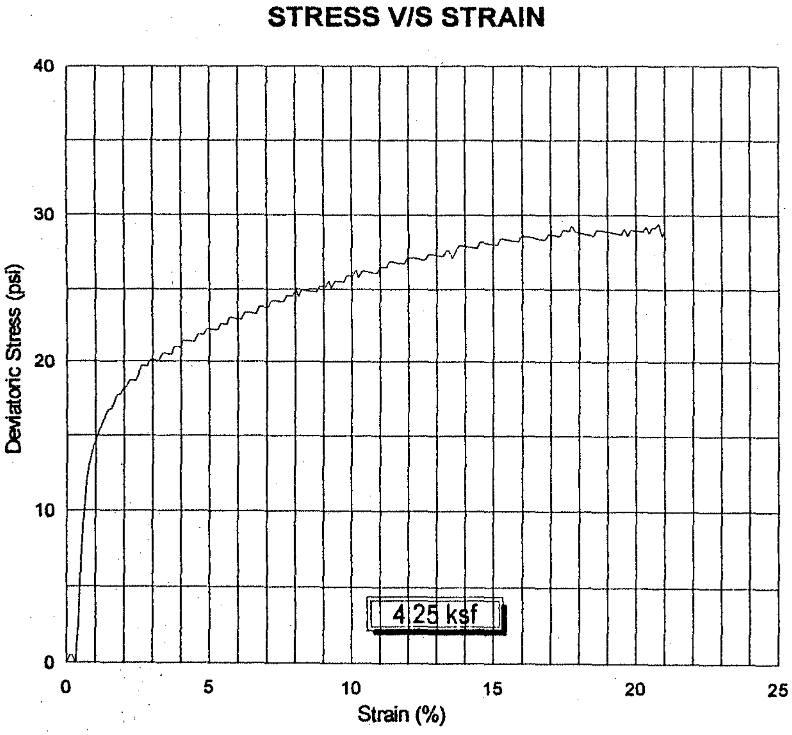


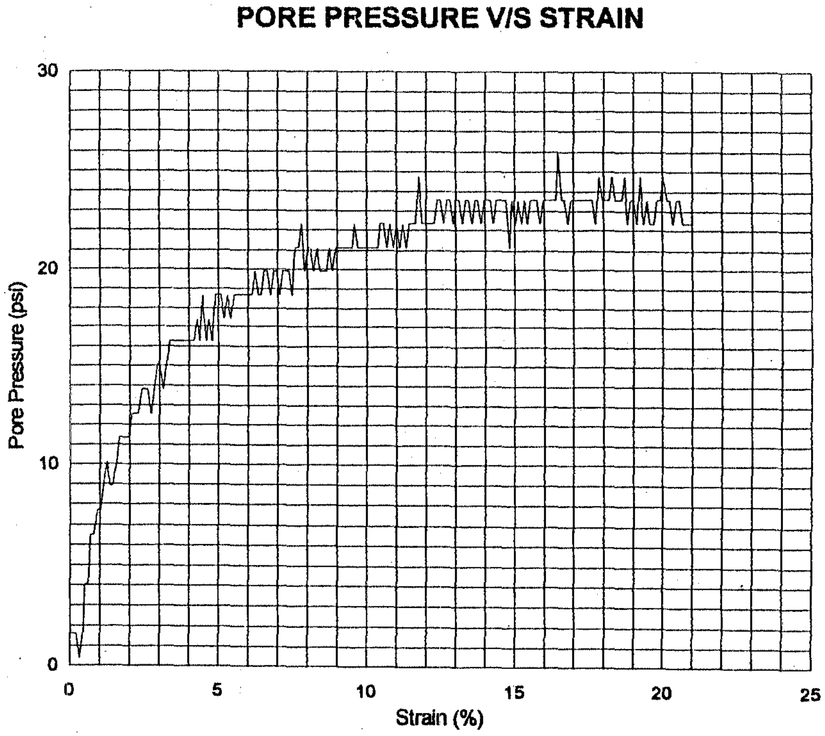
Height of Sample (inch)	5	Cell Pressure (psi)	10
Initial Diameter (inch)	2.416	Back Pressure (psi)	45
% Moisture - Before Test	21.2	Maximum Strength (psi))	37.63
% Moisture - After Test	20.4	Strain @ Failure (%)	11.61
Dry Density (pcf)	107.3	Pore Pressure @Failure (psi)	3.8
sigma3' (psi)=	6.2	sigma1' (psi)=	43.83

Boring No. B7 **Sample No.** 5B **Elev. or Depth (ft)** 21

Job Name : Premanente Creek **Job No.:** 99106.1 **Plate No:.** **B-4G**

CONSOLIDATED UNDRAINED COMPRESSION TEST





Height of Sample (inch)	5	Cell Pressure (psi)	30
Initial Diameter (inch)	2.416	Back Pressure (psi)	35
% Moisture - Before Test	24.8	Maximum Strength (psi))	29.50
% Moisture - After Test	23.2	Strain @ Failure (%)	20.79
Dry Density (pcf)	100.5	Pore Pressure @Failure (psi)	22.34

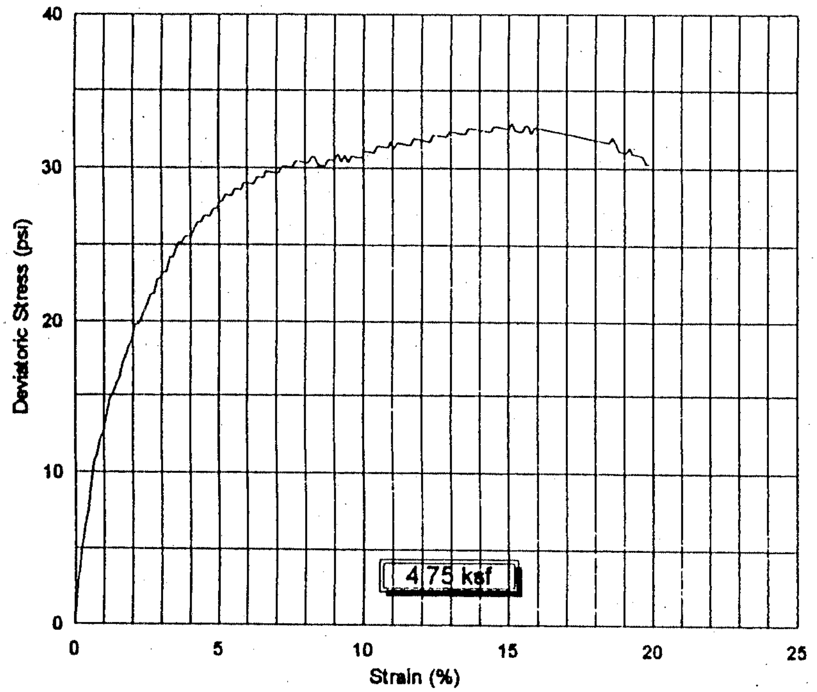
sigma3' (psi)= 7.66 sigma1' (psi)= 37.16

Boring No. B9 **Sample No.** 7A **Elev. or Depth (ft)** 32.5

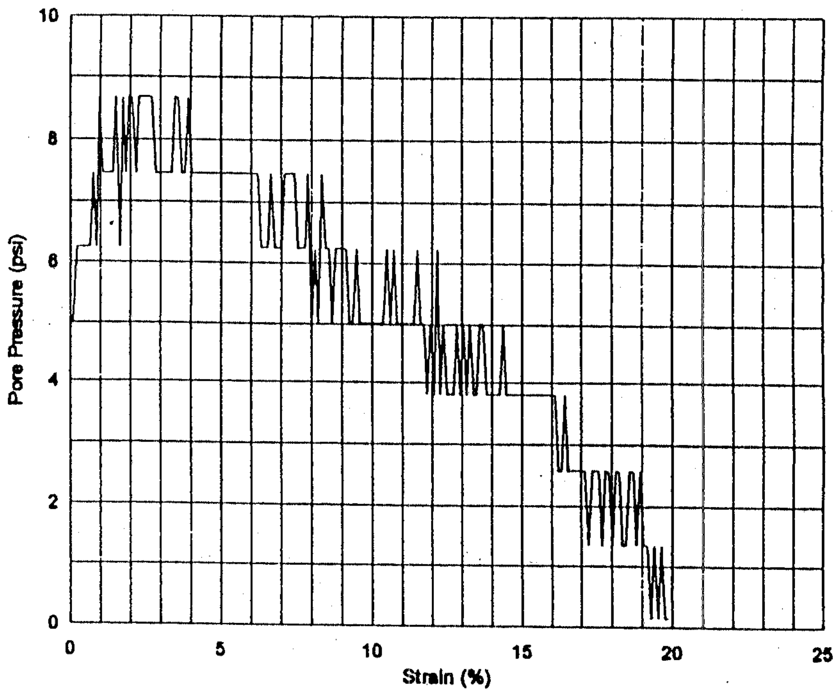
Job Name : Premanente Creek **Job No.:** 99106.1 **Plate No:.** B-4H

CONSOLIDATED UNDRAINED COMPRESSION TEST

STRESS V/S STRAIN



PORE PRESSURE V/S STRAIN



Height of Sample (inch)	5	Cell Pressure (psi)	10
Initial Diameter (inch)	2.416	Back Pressure (psi)	45
% Moisture - Before Test	11.4	Maximum Strength (psi))	32.96
% Moisture - After Test	14.8	Strain @ Failure (%)	15.14
Dry Density (pcf)	122.7	Pore Pressure @Failure (psi)	3.8

sigma3' (psi)= 6.2 sigma1' (psi)= 39.16

Boring No. B10 **Sample No.** 1B **Elev. or Depth (ft)** 2

Job Name : Permanente Creek Study **Job No.:** 999106.10 Plate No.: B-4I

Consolidated – Undrained
Triaxial Test Results for Sandy Lean & Lean Clays

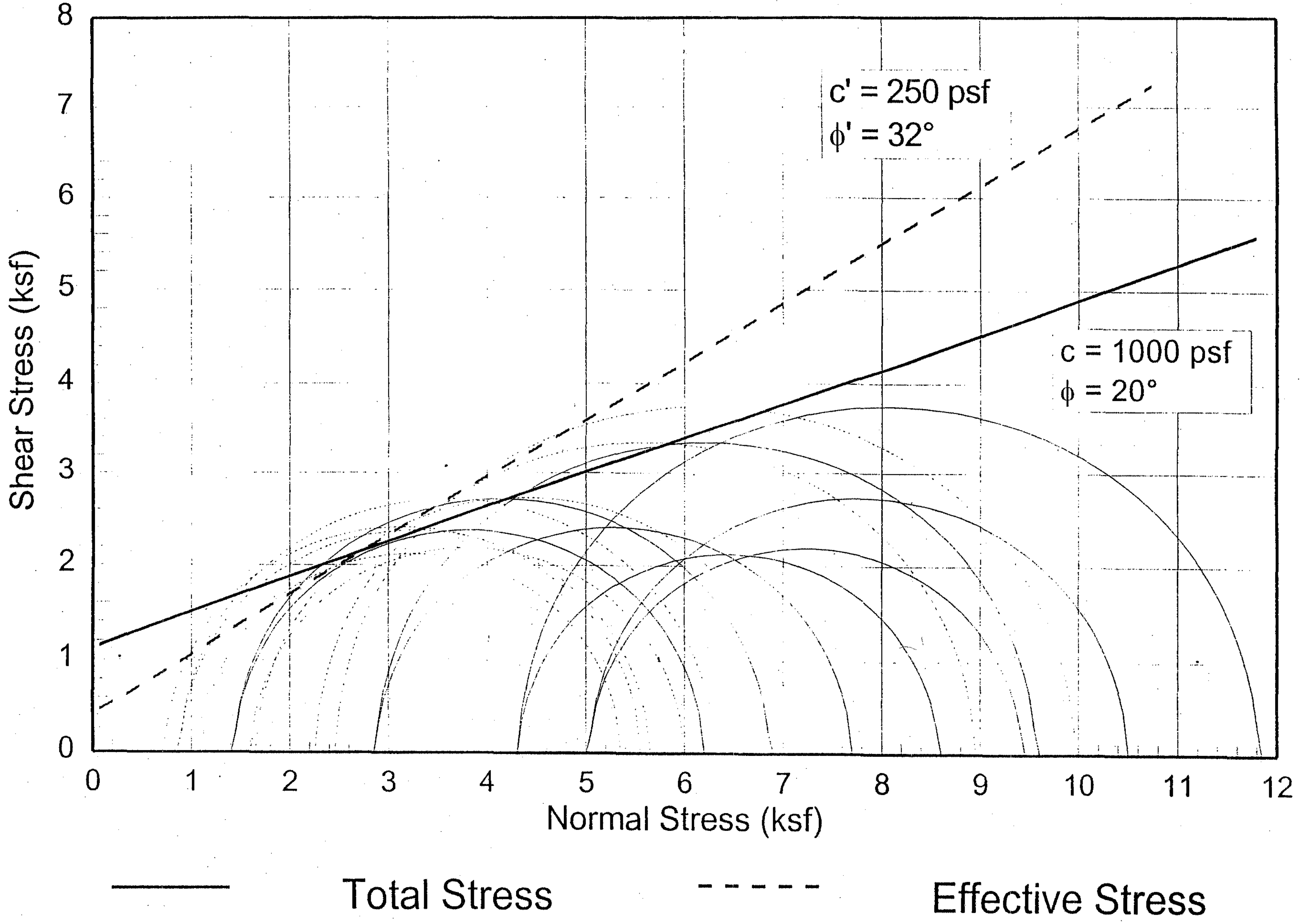
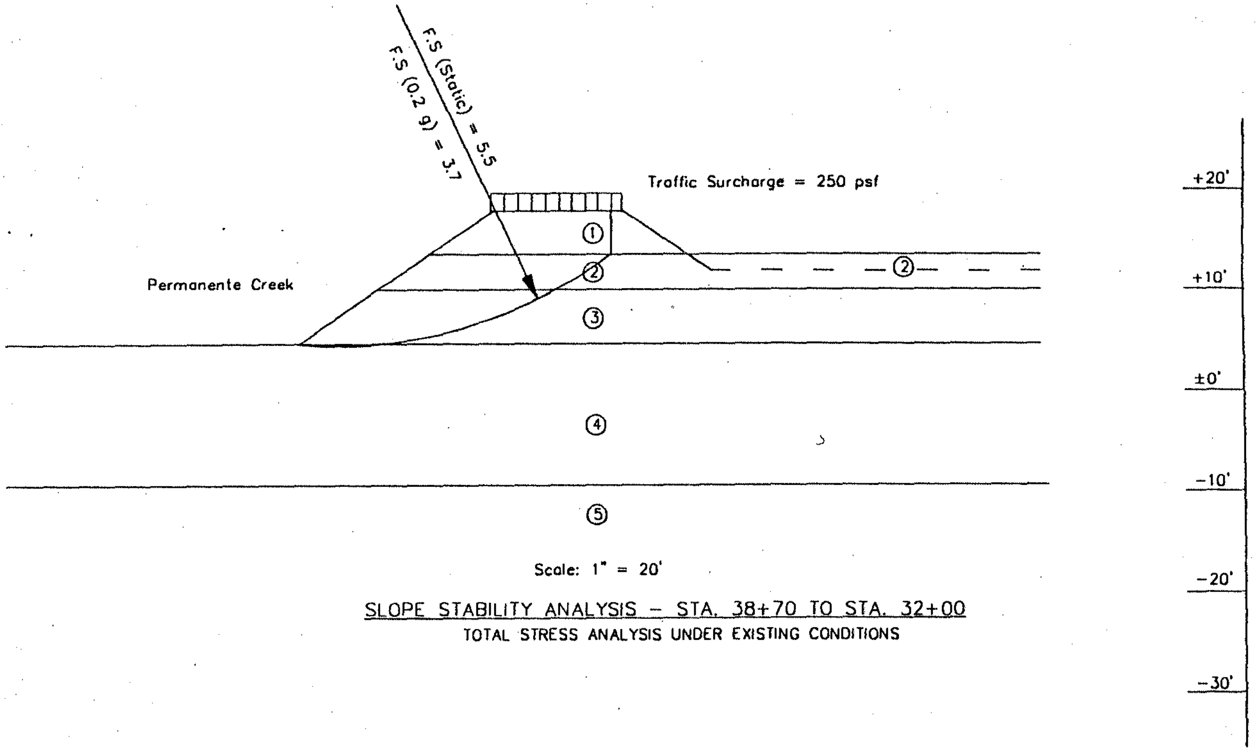


Plate B-4J

APPENDIX C

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Strength Parameters (Q or S)	
				c (psf)	Friction Angle
1	Sandy Fat Clay (CH)	120	125	2000	0
2	Lean Clay (CL)	120	125	2000	0
3	Silty Sand (SM)	120	125	0	35
4	Lean Clay w/ Sand (CL)	125	125	2500	0
5	Poorly graded Sand (SP)	125	125	0	38



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 38+70 TO STA. 32+00

TOTAL STRESS ANALYSIS UNDER EXISTING CONDITIONS

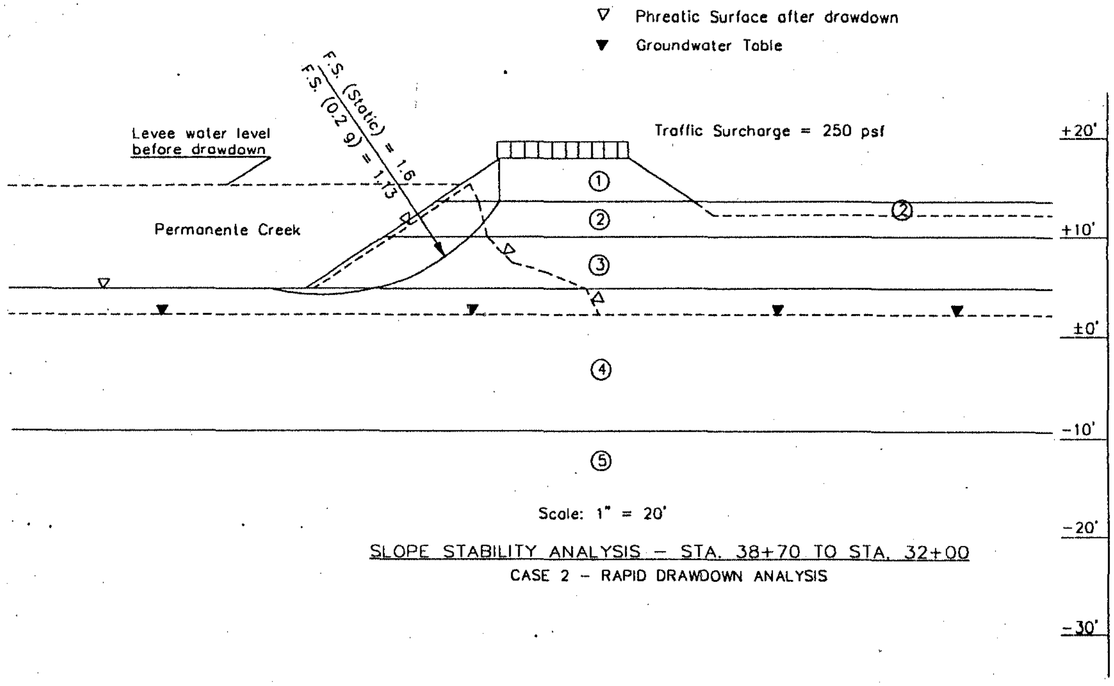
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-1

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Shear Strength Parameters			
				Strength Parameters (S)		Strength Parameters (R)	
				c (psf)	Friction Angle	c (psf)	Friction Angle
1	Sandy Fat Clay (CH)	120	125	250	28	750	20
2	Lean Clay (CL)	120	125	250	32	1000	20
3	Silty Sand (SM)	120	125	0	35	0	35
4	Lean Clay w/ Sand (CL)	125	125	250	32	1000	20
5	Poorly graded Sand (SP)	125	125	0	38	0	38



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 38+70 TO STA. 32+00
CASE 2 – RAPID DRAWDOWN ANALYSIS

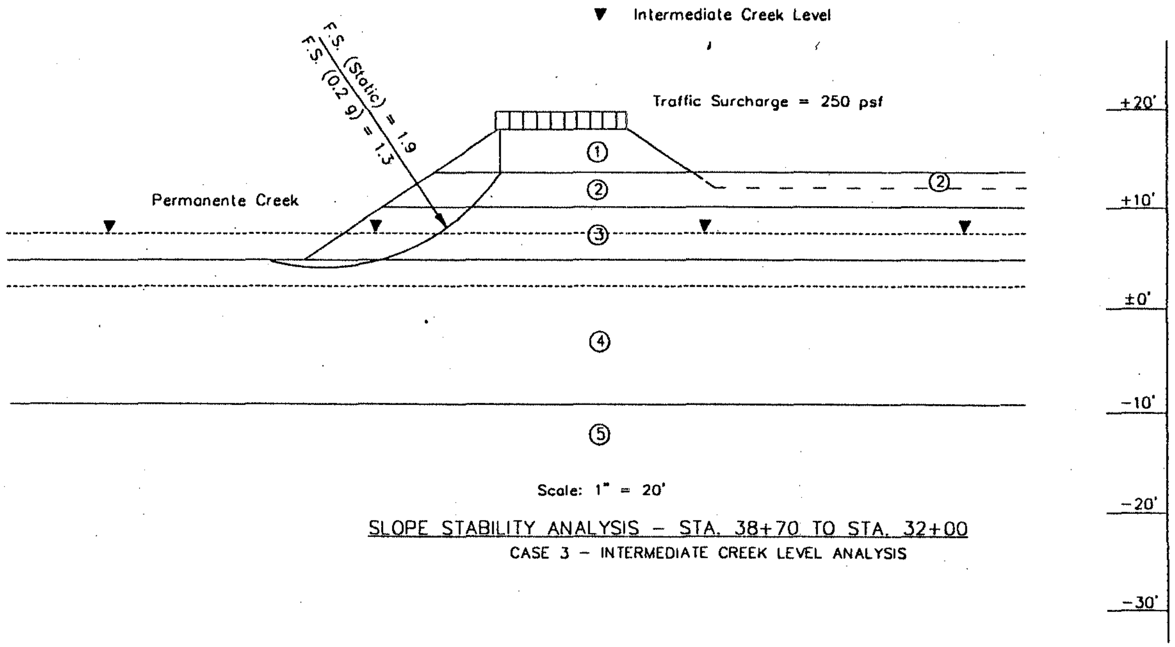
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-2

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Shear Strength Parameters			
				Strength Parameters (S)		Strength Parameters (R+S/2)	
				c (psf)	Friction Angle	c (psf)	Friction Angle
1	Sandy Fat Clay (CH)	120	125	250	28	500	24
2	Lean Clay (CL)	120	125	250	32	650	26
3	Silty Sand (SM)	120	125	0	35	0	35
4	Lean Clay w/ Sand (CL)	125	125	250	32	650	24
5	Poorly graded Sand (SP)	125	125	0	38	0	38



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS - STA. 38+70 TO STA. 32+00

CASE 3 - INTERMEDIATE CREEK LEVEL ANALYSIS

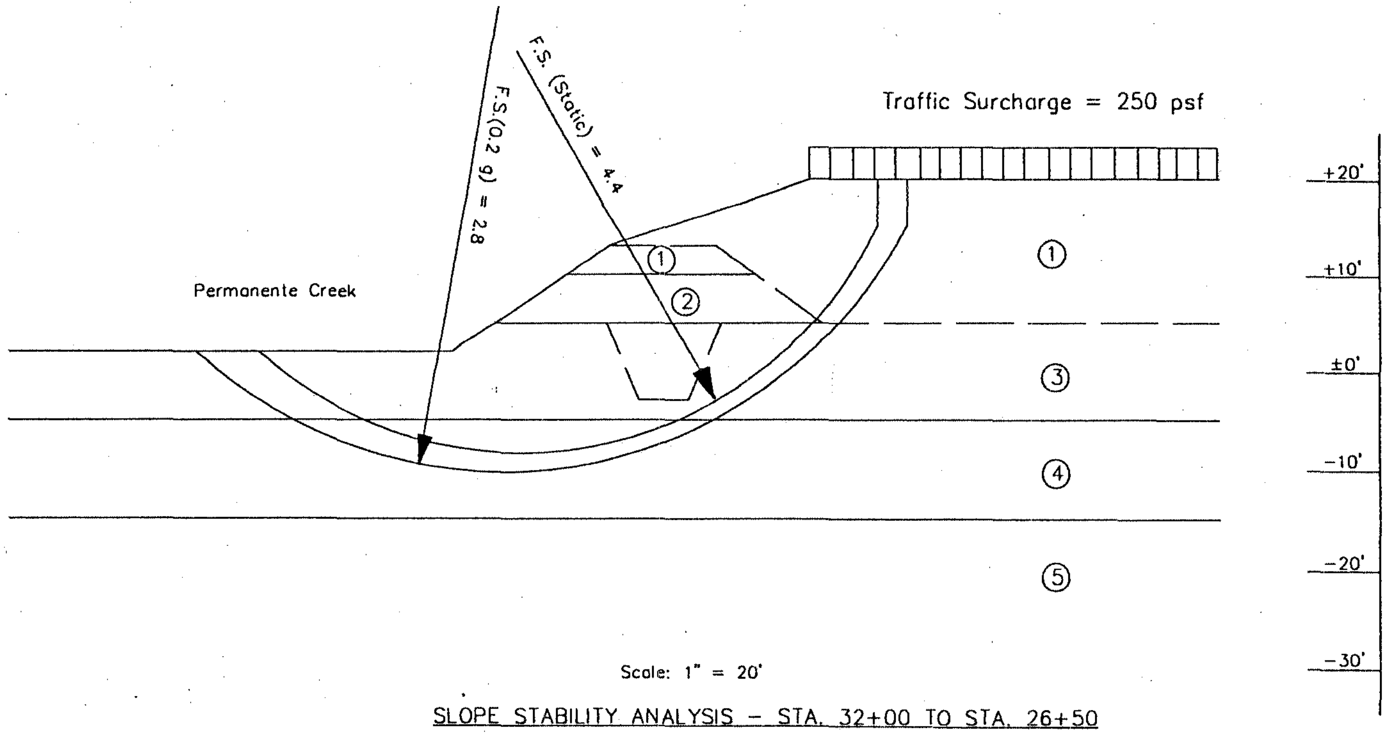
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-3

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Strength Parameters (Q or S)	
				c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	2500	0
2	Fat Clay (CH)	120	125	2500	0
3	Lean Clay (CL)	120	125	1500	0
4	Silty Sand (SM)	125	125	0	38
5	Lean Clay w/ sand (CL)	125	125	2500	0



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 32+00 TO STA. 26+50

TOTAL STRESS ANALYSIS UNDER EXISTING CONDITIONS

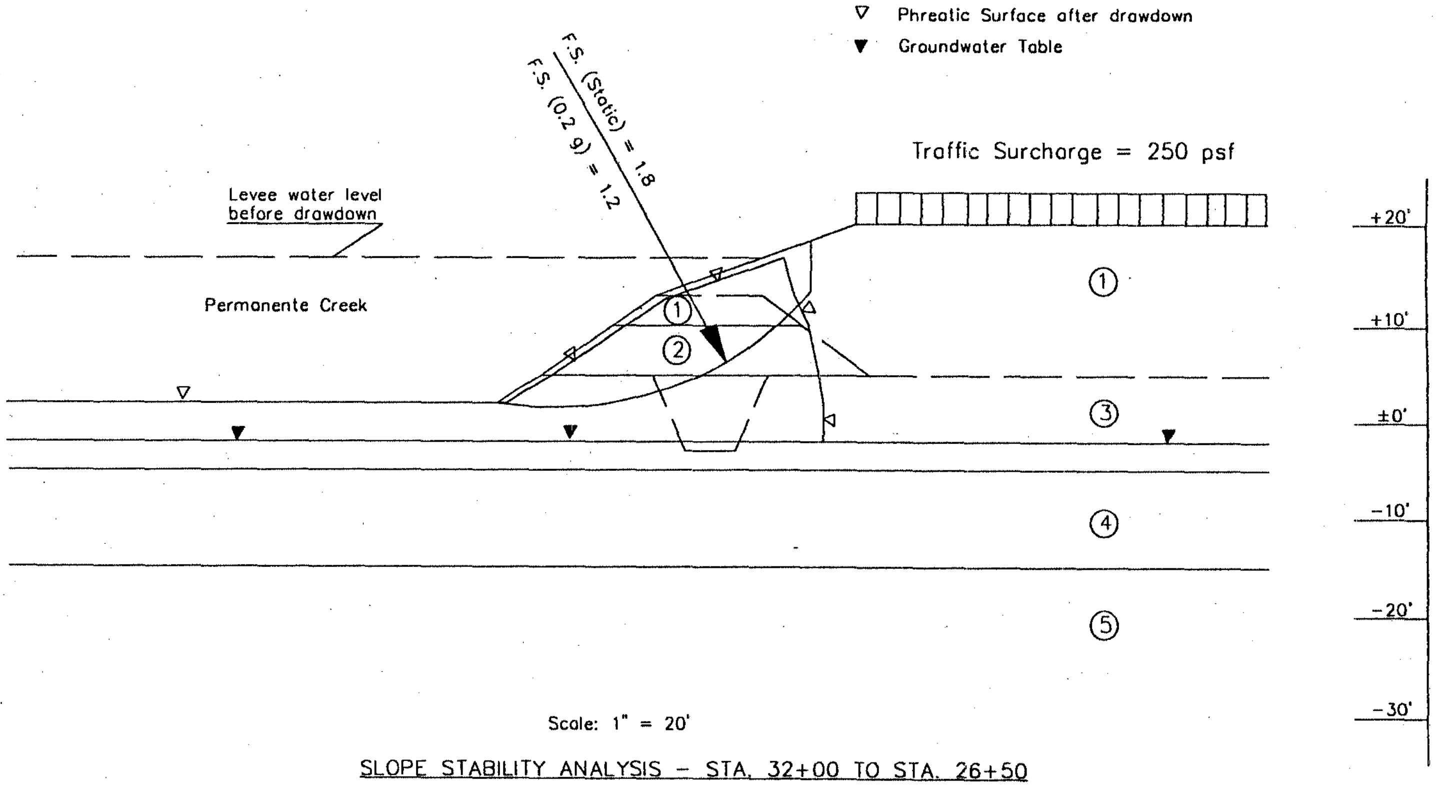
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-4

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Shear Strength Parameters			
				Strength Parameters (S)		Strength Parameters (R)	
				c (psf)	Friction Angle	c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	250	32	1000	20
2	Fat Clay (CH)	120	125	250	28	750	20
3	Lean Clay (CL)	120	125	250	32	1000	20
4	Silty Sand (SM)	125	125	0	38	0	38
5	Lean Clay w/ sand (CL)	125	125	250	32	1000	20



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 32+00 TO STA. 26+50

CASE 2 – RAPID DRAWDOWN ANALYSIS

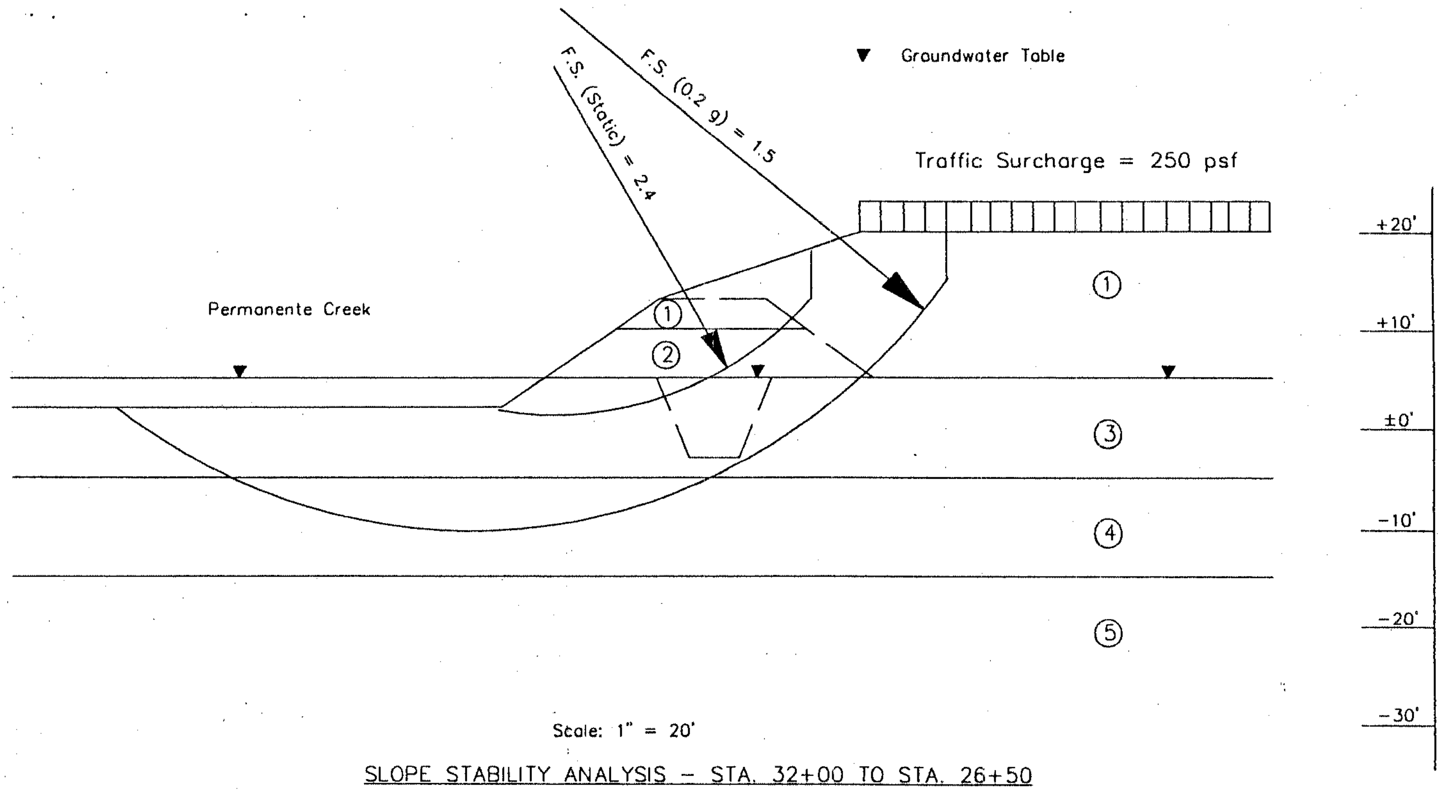
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-5

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Shear Strength Parameters			
				Strength Parameters (S)		Strength Parameters (R+S/2)	
				c (psf)	Friction Angle	c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	250	32	650	26
2	Fat Clay (CH)	120	125	250	28	500	24
3	Lean Clay (CL)	120	125	250	32	650	26
4	Silty Sand (SM)	125	125	0	38	0	38
5	Lean Clay w/ sand (CL)	125	125	250	32	650	26



SLOPE STABILITY ANALYSIS – STA. 32+00 TO STA. 26+50

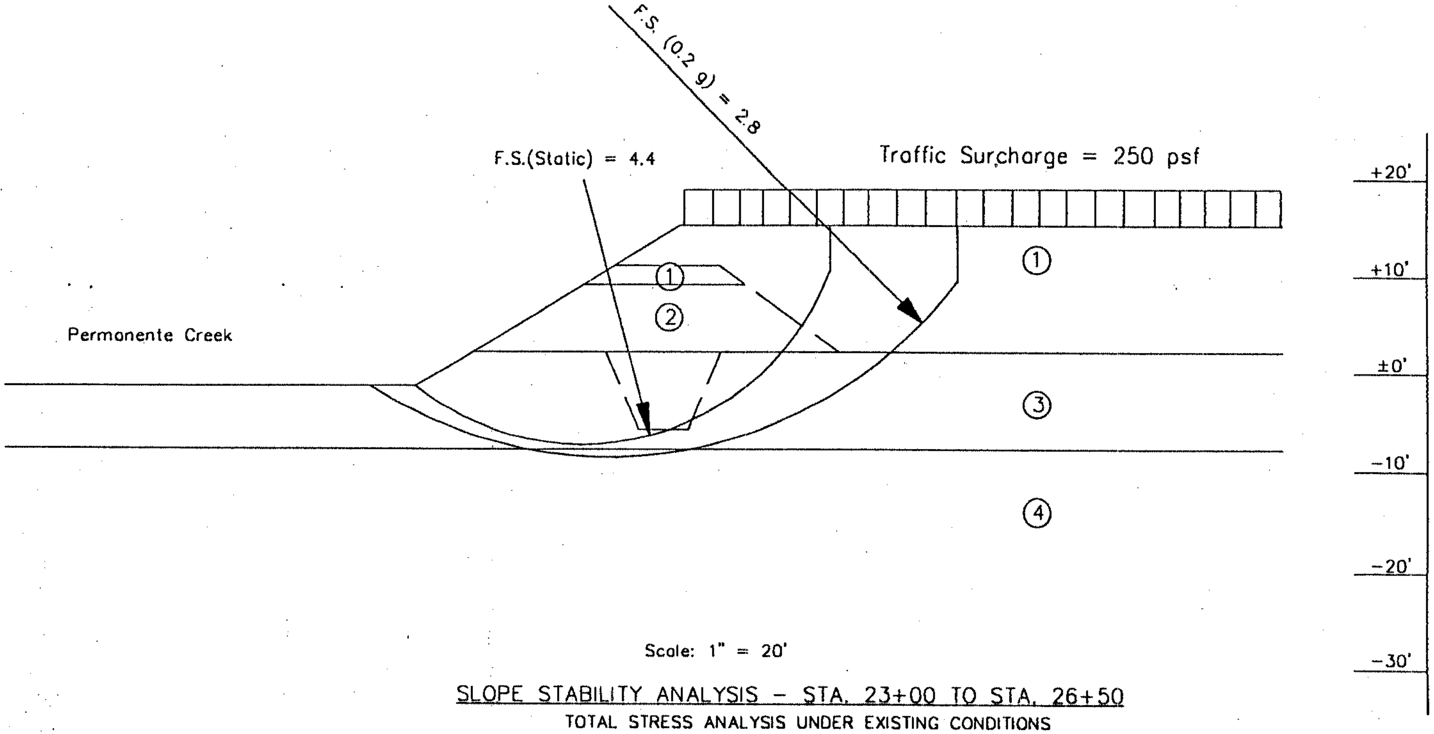
CASE 3 – INTERMEDIATE CREEK LEVEL ANALYSIS

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10	PLATE NO.: C-6

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Strength Parameters (Q or S)	
				c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	2500	0
2	Fat Clay (CH)	120	125	2500	0
3	Fat Clay (CH)	120	120	1500	0
4	Lean Clay (CL)	125	125	3000	0



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 23+00 TO STA. 26+50

TOTAL STRESS ANALYSIS UNDER EXISTING CONDITIONS

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

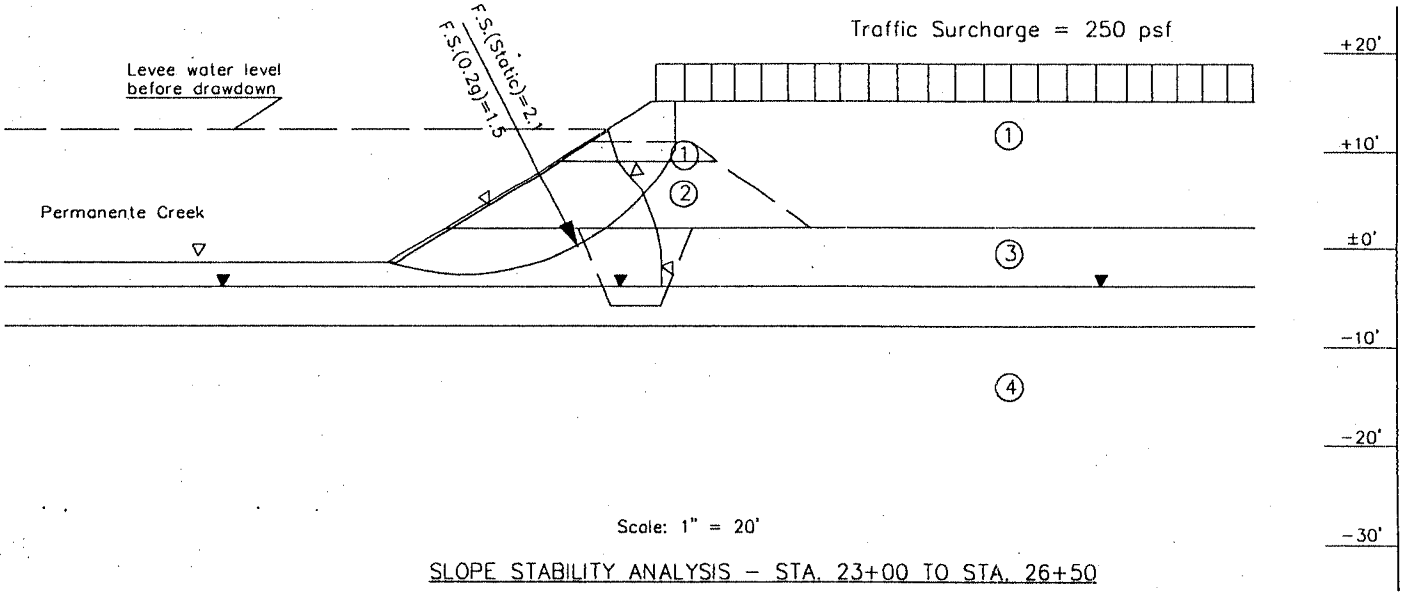
PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-7

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Shear Strength Parameters			
				Strength Parameters (S)		Strength Parameters (R)	
				c (psf)	Friction Angle	c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	250	32	1000	20
2	Fat Clay (CH)	120	125	250	28	750	20
3	Fat Clay (CH)	120	120	250	28	750	20
4	Lean Clay (CL)	125	125	250	32	1000	20

▽ Phreatic Surface after drawdown
▼ Groundwater Table



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 23+00 TO STA. 26+50
CASE 2 – RAPID DRAWDOWN ANALYSIS

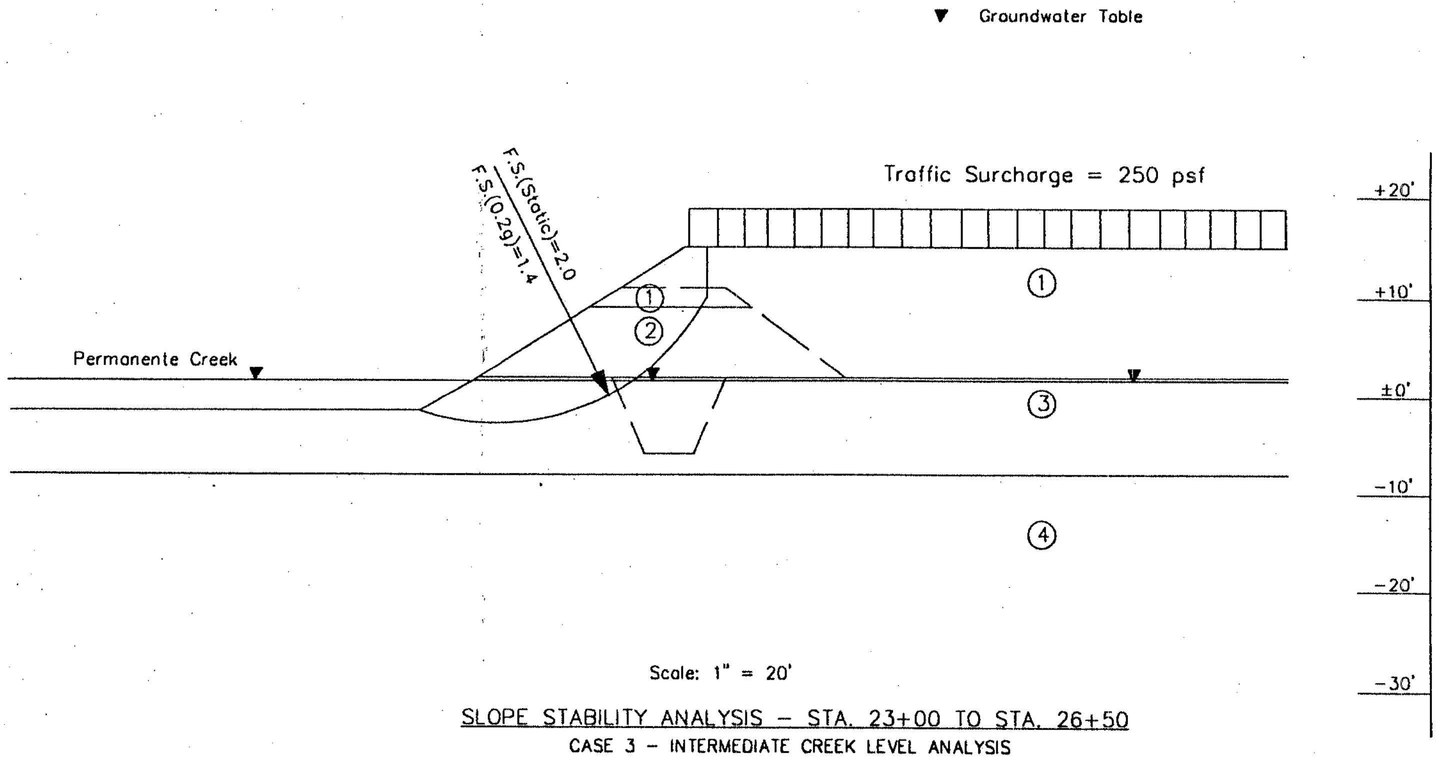
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-8

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Shear Strength Parameters			
				Strength Parameters (S)		Strength Parameters (R+s/2)	
				c (psf)	Friction Angle	c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	250	32	650	26
2	Fat Clay (CH)	120	125	250	28	500	24
3	Fat Clay (CH)	120	120	250	28	500	24
4	Lean Clay (CL)	125	125	250	32	650	26



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 23+00 TO STA. 26+50

CASE 3 – INTERMEDIATE CREEK LEVEL ANALYSIS

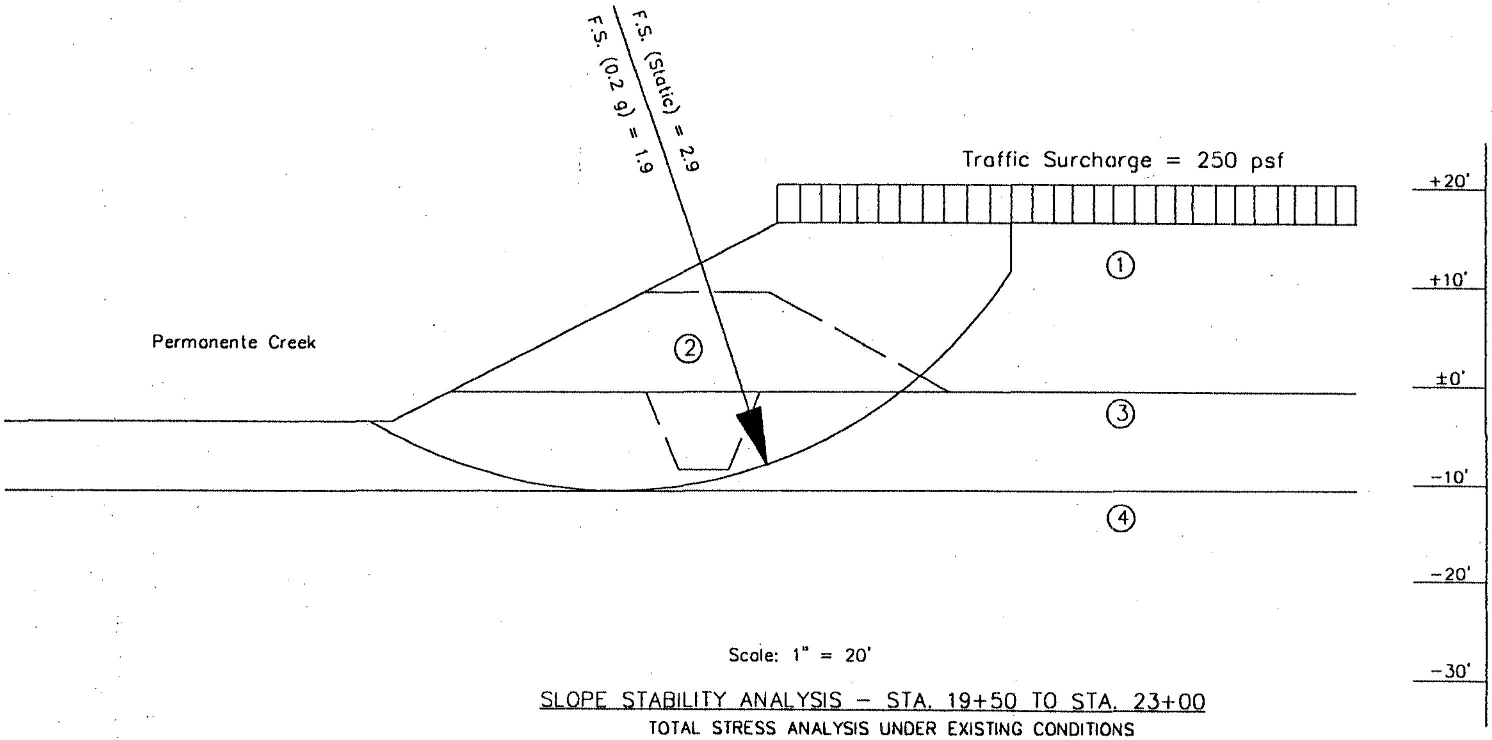
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-9

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Strength Parameters (Q or S)	
				c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	2500	0
2	Fat Clay (CH)	120	125	2500	0
3	Bay Mud (OH)	95	95	900	0
4	Lean Clay (CL)	125	125	3000	0



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS − STA. 19+50 TO STA. 23+00

TOTAL STRESS ANALYSIS UNDER EXISTING CONDITIONS

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

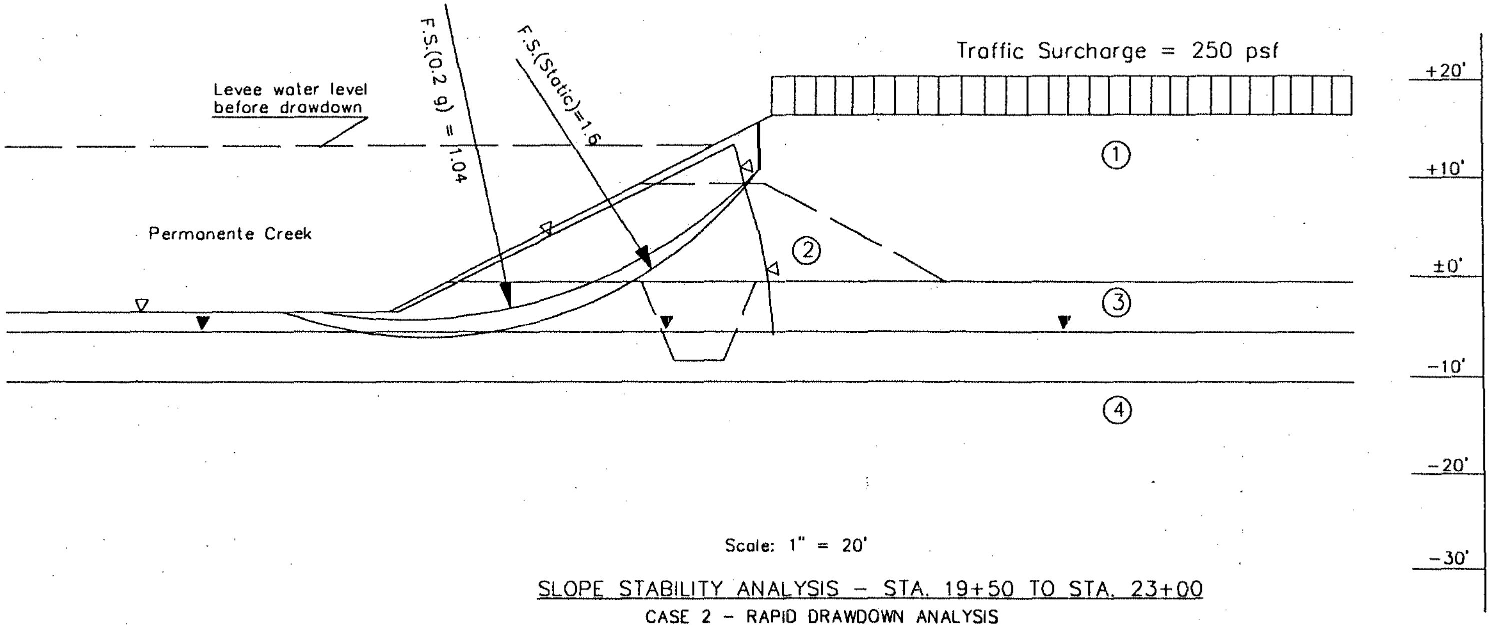
PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-10

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Shear Strength Parameters			
				Strength Parameters (S)		Strength Parameters (R)	
				c (psf)	Friction Angle	c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	250	32	1000	20
2	Fat Clay (CH)	120	125	250	28	750	20
3	Bay Mud (OH)	95	95	100	26	250	20
4	Lean Clay (CL)	125	125	250	32	1000	20

▽ Phreatic Surface after drawdown
▼ Groundwater Table



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 19+50 TO STA. 23+00

CASE 2 – RAPID DRAWDOWN ANALYSIS

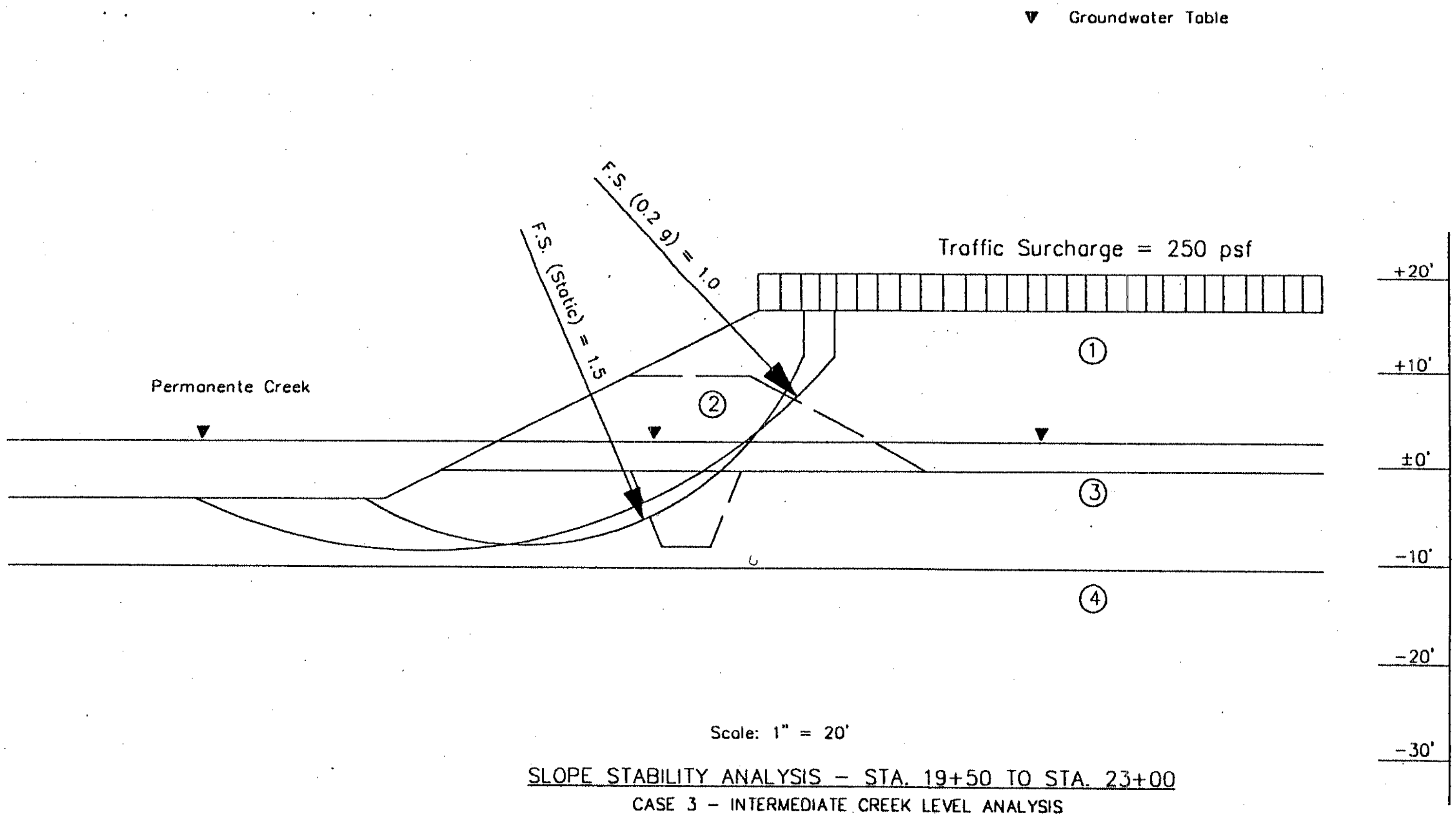
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-11

Soil No.	Soil Description	Moist Unit Weight (pcf)	Total Unit Weight (pcf)	Shear Strength Parameters			
				Strength Parameters (S)		Strength Parameters (R+S/2)	
				c (psf)	Friction Angle	c (psf)	Friction Angle
1	Sandy Lean Clay (CL)	120	125	250	32	650	26
2	Fat Clay (CH)	120	125	250	28	500	24
3	Bay Mud (OH)	95	95	100	26	175	23
4	Lean Clay (CL)	125	125	250	32	650	26



Scale: 1" = 20'

SLOPE STABILITY ANALYSIS – STA. 19+50 TO STA. 23+00

CASE 3 – INTERMEDIATE CREEK LEVEL ANALYSIS

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PERMANANETE CREEK LEVEES CERIFICATION PROJECT
CITY OF MOUNTAIN VIEW, CA

JOB NO.: 99106.10

PLATE NO.: C-12

APPENDIX D

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using track-mounted, rotary-wash drilling equipment. Three 4 7/8-inch-diameter exploratory borings were drilled on March 17, 1999, to a maximum depth of 30 feet. The approximate locations of the exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D-2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of borings were approximately determined by pacing from existing site boundaries. Elevations of the borings were determined by interpolation from plan contours. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D-1586). In addition, 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Relatively undisturbed samples were also obtained with 3.0-inch I.D. Shelby Tube Samplers, which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the undrained shear strength of soil samples using a Torvane device, and the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depth.

The attached boring logs and related information depict subsurface conditions only at the locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * *

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW	●	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	○	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	○	Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC	▨	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW	⁘	Well graded sands, gravelly sands, little or no fines
			SP	⁘	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	▥	Silty sands, sand-silt-mixtures, non-plastic fines
			SC	▨	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50 %		ML	▥	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	☰	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH	▥	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	▨	Inorganic clays of high plasticity, fat clays
			OH	≋	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	⩛	Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

200 — 40 — 10 — 4 — 3/4" — 3" — 12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

☒	TERZAGHI SPLIT SPOON STANDARD PENETRATION
⧗	MODIFIED CALIFORNIA
▯	D & M UNDERWATER SAMPLER
■	SHELBY TUBE
○	NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

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FIGURE A-1

EXPLORATORY BORING: EB-1

Sheet 1 of 1

DRILL RIG: FAILING 250
BORING TYPE: ROTARY WASH
LOGGED BY: LCK
START DATE: 3-17-99 FINISH DATE: 3-17-99

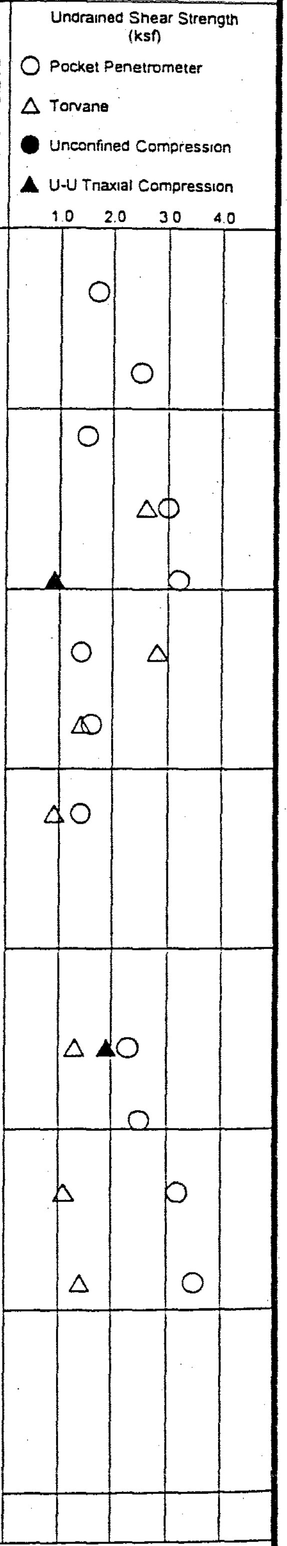
PROJECT NO: 1026-3C
PROJECT: ALZA CORPORATE OFFICES
LOCATION: MOUNTAIN VIEW
COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 12.1 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
12.1	0	SILTY CLAY (CL/CH) [FILL] stiff, moist, brown and black, moderate and high plasticity, trace sand and fine gravel	CL-CH	12	20	103	
				300psi	21	107	
	5			18	9		
				25	22	101	
		increasing gravel					
2.5	10	SANDY CLAY (CL) stiff, moist, light gray, moderate to high plasticity, trace sand	CL	500psi	24	100	
		color grades to brown		24	22	102	
-1.9		SILTY CLAY (CL) stiff, moist, light brown, moderate plasticity	CL	11	25	103	
	15			250psi	22	105	
-6.1		CLAYEY SAND (SC) medium dense, wet, brown, low to moderate plasticity, very fine to fine sand	SC	150psi			
	20			500psi			
-9.7		SILTY CLAY (CL) stiff, moist, brown, moderate plasticity color grades to greenish gray	CL	20	30	96	
	25	very stiff, color grades to brown		42	27	103	
				34	25	104	
		color grades to greenish gray		55	24	105	
17.9	30	Bottom of Boring @ 30 feet.					
	35						



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTORY WASH METHOD OF DRILLING

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EXPLORATORY BORING: EB-2

Sheet 1 of 1

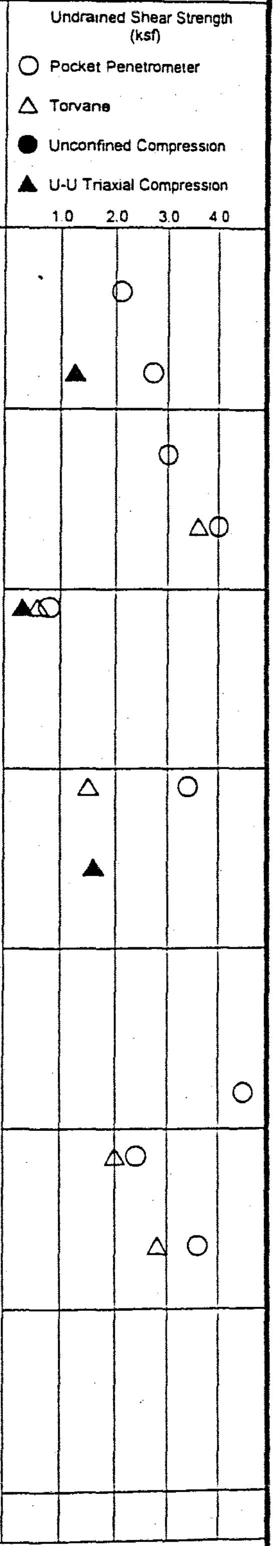
DRILL RIG: FAILING 250
BORING TYPE: ROTARY WASH
LOGGED BY: LCK
START DATE: 3-17-99 FINISH DATE: 3-17-99
PROJECT NO: 1026-3C
PROJECT: ALZA CORPORATE OFFICES
LOCATION: MOUNTAIN VIEW
COMPLETION DEPTH: 29.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 12.3 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
12.3	0	SILTY CLAY (CL/CH) [FILL] stiff, moist, brown and black, moderate and high plasticity, trace sand, fine gravel	CL-CH	16	22	91	
				400psi	21	87	
	5	Average vertical permeability = 2x10E-5 cm/s					
		very stiff		400psi	25	92	
		Average vertical permeability = 2x10E-6 cm/s					
		color grades to black, high plasticity		25	25	100	
		trace wood					
2.1	10	SANDY CLAY (CL) medium stiff, moist, brown, moderate plasticity	CL	350psi	28	92	
		wet		400psi			
	15			400psi			
				400psi	24	102	
-6.7	20	SAND (SP-SC) dense, brown, fine to medium sand, trace clay	SP-SC	40	12		
		increasing fine gravel					
-9.9		SILTY CLAY (CL) hard, moist, greenish gray	CL	7	10		
				400psi			
	25	very stiff					
		color grades to olive brown		35	25	104	
				54	25	103	
-16.7	30	Bottom of Boring @ 29 feet.					
	35						



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTORY WASH METHOD OF DRILLING

EXPLORATORY BORING: EB-3

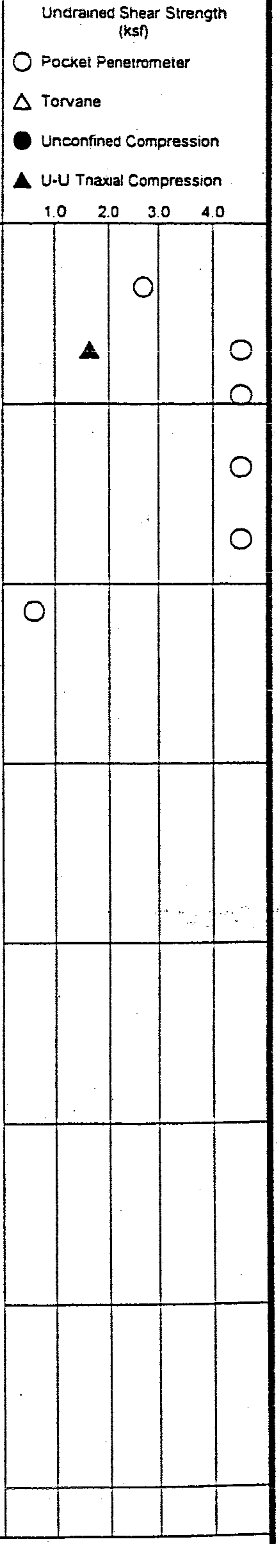
Sheet 1 of 1

DRILL RIG: FAILING 250
BORING TYPE: ROTARY WASH
LOGGED BY: LCK
START DATE: 3-17-99 FINISH DATE: 3-17-99

PROJECT NO: 1026-3C
PROJECT: ALZA CORPORATE OFFICES
LOCATION: MOUNTAIN VIEW
COMPLETION DEPTH: 13.5 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION: 13.7 FT. (+/-)	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
13.7	0		SILTY CLAY (CL) [FILL] stiff, moist, dark brown, moderate plasticity, trace fine to medium sand, gravel, roots		16		15	101	
					400psi		11	107	
	5		Average vertical permeability = 2x10E-5 cm/s very stiff to hard, increasing sand, increasing gravel	CL	33		12	103	
					24		14	106	
			sand seam at sample bottom color grading to black, moderate to high plasticity		32		14	112	
3.7	10		SANDY CLAY (CL) medium stiff, brown, fine to medium sand, trace gravel sand seam	CL	6		18	102	
1.4			SAND (SP) dense, wet, brown, fine to coarse sand, trace fine to medium gravel		52		7		
0.2	15		Bottom of Boring @ 13.5 feet.						
	20								
	25								
	30								
	35								



GROUND WATER OBSERVATIONS:
NOT APPLICABLE DUE TO ROTORY WASH METHOD OF DRILLING

LA CORP GDT 5/11/99 MV' DGB

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APPENDIX B
LABORATORY INVESTIGATION

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D-2216) on 29 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D-2937) were performed on 25 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

UU-Triaxial Compression: Unconsolidated-undrained triaxial compression tests (ASTM D2850) were performed on seven undisturbed samples of the clayey subsurface soils to evaluate the undrained shear strengths of these materials. Failure was taken as the peak normal stress. Results of these tests are presented on the boring logs at the appropriate sample depths.

CU-Triaxial Compression: Consolidated-undrained triaxial compression tests with pore pressure measurements (ASTM D4767) were performed on three undisturbed samples of the clayey fill soils and three undisturbed samples of the native clayey soils to evaluate the undrained shear strengths of these materials. Failure was taken as the peak normal stress. Results of these tests are presented on the boring logs at the appropriate sample depths.

Permeability: Constant head permeability tests (ASTM D5084) were performed on three undisturbed samples of the existing clayey fill soils to estimate the coefficient of permeability. Results of these tests are presented on the boring logs at the appropriate sample depths .

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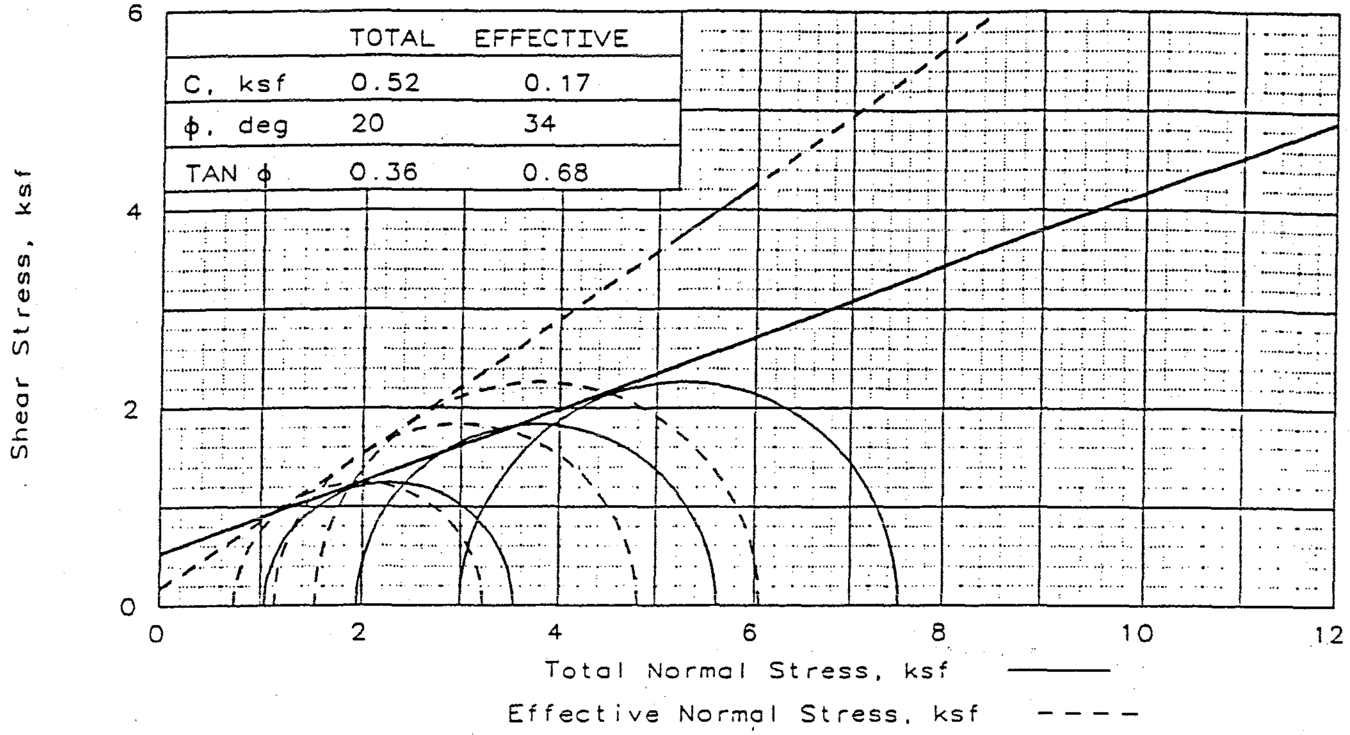
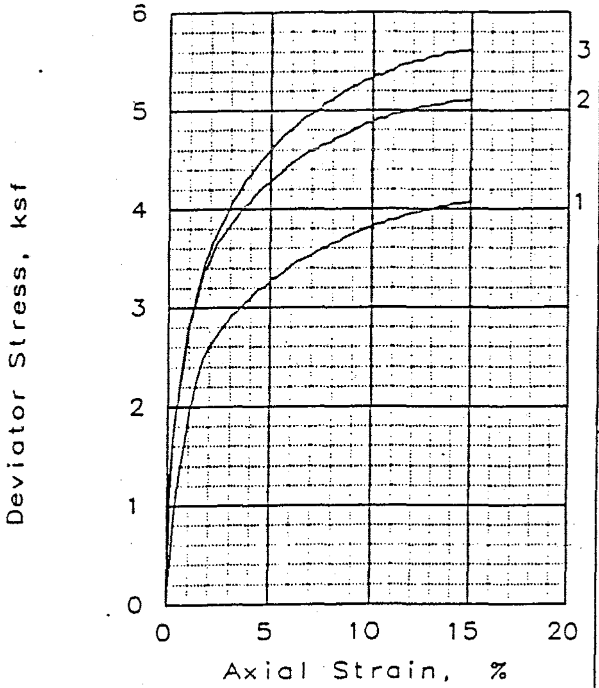


	TOTAL	EFFECTIVE
C, ksf	0.52	0.17
ϕ, deg	20	34
TAN ϕ	0.36	0.68



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	21.8	22.9	22.7
	DRY DENSITY, pcf	106.9	104.1	105.0
	SATURATION, %	96.1	94.2	95.7
	VOID RATIO	0.635	0.679	0.664
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	6.00	6.00	6.00
AT TEST	WATER CONTENT, %	22.0	22.8	21.7
	DRY DENSITY, pcf	108.2	106.7	108.7
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.615	0.638	0.609
	DIAMETER, in	2.86	2.84	2.83
	HEIGHT, in	5.98	5.96	5.95
	Strain rate, %/min	0.03		0.03
	EFF CELL PRESSURE, ksf	1.04	1.94	3.00
	Deviator Stress, ksf	2.49	3.67	4.53
	EXCESS PORE PR., ksf	0.30	0.81	1.47
	STRAIN, %	1.7	2.4	4.6
	ULT. STRESS, ksf			
	EXCESS PORE PR., ksf			
	STRAIN, %			
	$\bar{\sigma}_1$ FAILURE, ksf	3.23	4.81	6.05
	$\bar{\sigma}_3$ FAILURE, ksf	0.73	1.14	1.53

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: olive brown
clayey SAND

ASSUMED SPECIFIC GRAVITY= 2.8
REMARKS: Strengths taken at the
peak stress ratios.

Fig. No.: ______

CLIENT: Lowney

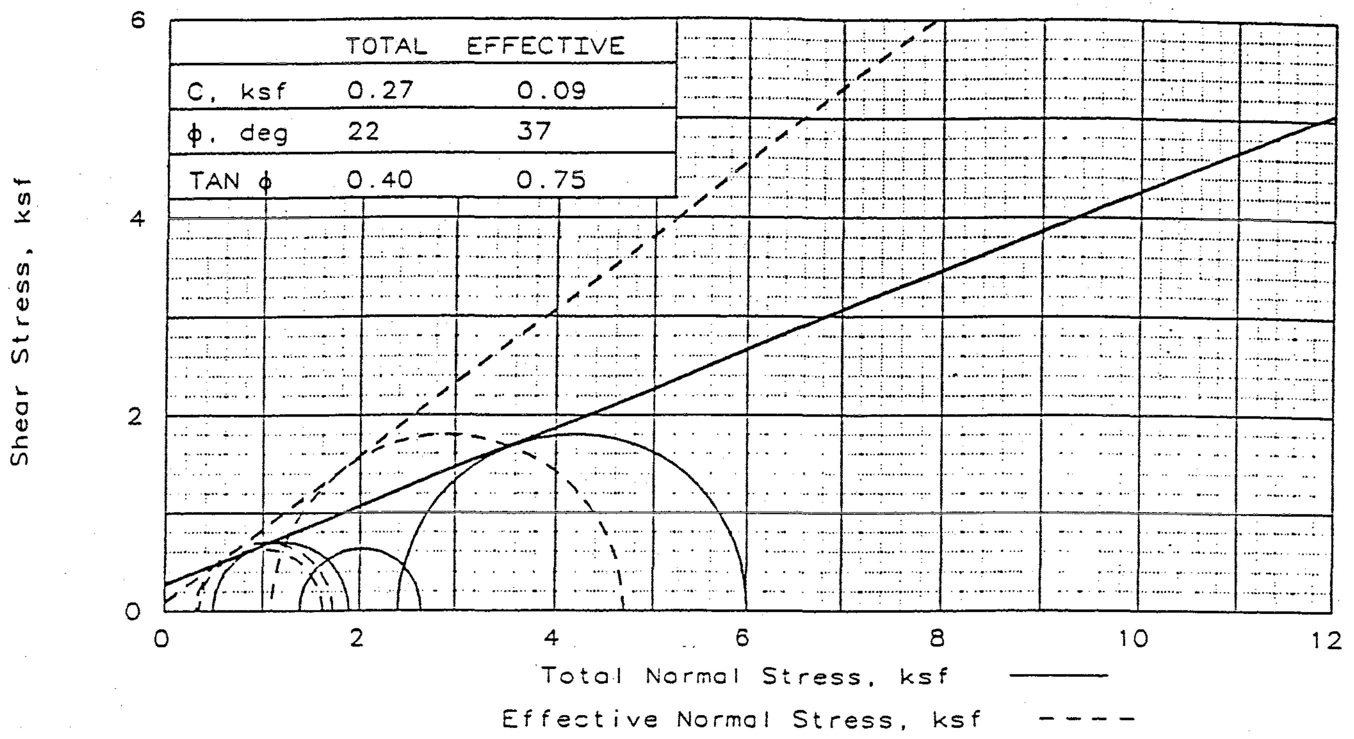
PROJECT: 1026-3C

SAMPLE LOCATION: EB-1 @ 17.5'

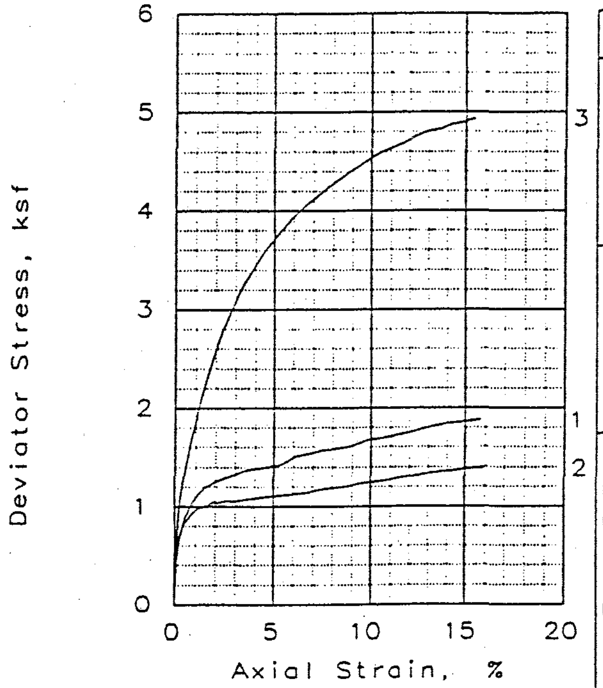
PROJ. NO.: 028-617 DATE: 3/25/99

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY



	TOTAL	EFFECTIVE
C, ksf	0.27	0.09
ϕ, deg	22	37
TAN ϕ	0.40	0.75



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	22.0	20.8	15.7
	DRY DENSITY, pcf	98.0	99.8	113.4
	SATURATION, %	82.7	81.3	87.0
	VOID RATIO	0.719	0.689	0.487
	DIAMETER, in	2.87	2.87	2.87
	HEIGHT, in	5.75	5.75	6.00
AT TEST	WATER CONTENT, %	23.6	23.5	17.3
	DRY DENSITY, pcf	103.0	103.1	114.9
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.637	0.635	0.467
	DIAMETER, in	2.81	2.85	2.87
	HEIGHT, in	5.72	5.66	5.90
	Strain rate, %/min	0.03	0.03	0.03
	EFF CELL PRESSURE, ksf	0.49	1.38	2.40
	Deviator Stress, ksf	1.40	1.27	3.60
	EXCESS PORE PR., ksf	0.17	1.04	1.31
	STRAIN, %	4.5	11.1	4.5
	ULT. STRESS, ksf			
	EXCESS PORE PR., ksf			
	STRAIN, %			
	$\bar{\sigma}_1$ FAILURE, ksf	1.72	1.61	4.70
	$\bar{\sigma}_3$ FAILURE, ksf	0.32	0.35	1.09

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: undisturbed
DESCRIPTION: brown clayey SAND
w/gravel

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: The material varied
slightly from sample to samp.
Strengths taken at the peak
stress ratios.
Fig. No.: ______

CLIENT: Lowney

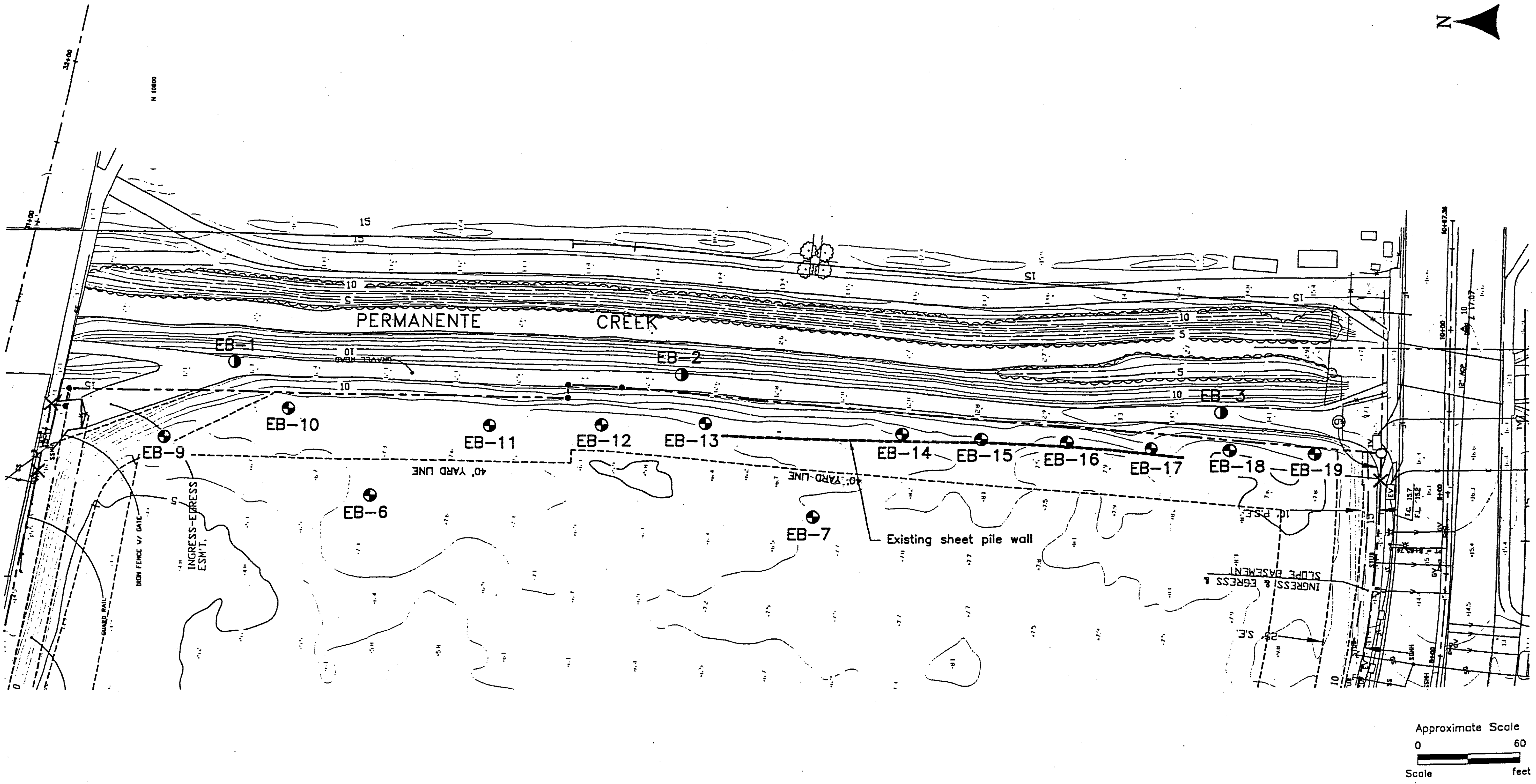
PROJECT: 1026-3C

SAMPLE LOCATION: EB1-3 @ 5'

PROJ. NO.: 028-618 DATE: 3/26/99

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY



LEGEND

◑ – Approximate location of exploratory boring

⊕ – Approximate location of previous exploratory boring (5/97)

Base by Mark Thomas & Co.

SITE PLAN
ALZA CORPORATE OFFICES
Mountain View, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 2

1026-3C

5/99-EB